HYDRO-ELECTRIC PRACTICE

VON SCHON

HYDRO-ELECTRIC PRACTICE

A PRACTICAL MANUAL OF THE DEVELOPMENT OF WATER POWER, ITS CONVERSION TO ELECTRIC ENERGY, AND ITS DISTANT TRANSMISSION

BY

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PREFACE TO SECOND EDITION

THE fact that the first edition of this treatise was exhausted in eight months is proof of an existing demand for it which exceeds any expectation of the author and which prompts him to send forth this second revised edition.

The revision consists mainly of a detailed treatment of the market, flow discussion, pondage, and storage in Part I.; of development scope and equipment in Part II.; and of Part III. "*Operating and Maintaining the Plant.*" The tables of rivers' drainage areas and low run-off, of navigable rivers, and the forms of Government permits and licenses have been taken out to make room for the above more valuable matter and because this information may now be readily obtained from Government publications.

No corrections have been made in cost estimates of works, equipment, and operation; those given are approximately correct for the conditions prevailing in the United States during 1907. The reader can readily make the proper allowances for changes in prices of materials and labor. The same holds good for quotations of current values.

In its revised condition "Hydro-electric Practice" is now presented anew in what is believed to be a more complete and useful treatment.

H. A. E. C. VON SCHON.

DETROIT, MICH., March, 1911.

PREFACE

The economical transmission of electric energy to distances great and small, the rapidly increasing utilization of electro-motive power in industrial establishments, and the advent of the electric interurban railroads are responsible for the marked movement of impressing water-powers to the service of generating electric current; and now water-power, which had been almost relegated to obscurity by the perfection of the steam-engine, is not only regaining but even exceeding its former importance as an economical prime power source.

It is entirely within the facts to state that a normally conditioned hydro-electric power plant can successfully compete with the most refined steam-power plant and the lowest priced fuel, natural gas.

No wonder then that water-powers are to-day being sought after with feverish activity, and that some remarkable successes have been achieved, but also that many disastrous failures must be recorded.

Hydro-electric power development is a much more complex undertaking than a large majority of the promoters of such enterprises realize when the subject is first approached, but which is most forcibly impressed upon them when the carrying out of the project is seriously attempted. Unfortunately, the most dangerous pitfalls are encountered at the beginning of the undertaking, and unless these are properly guarded against the finished work may disclose some incurable defects.

Developments of the important natural resources of mines, of forests, and of manufacturing and transportation projects are rarely undertaken except upon the findings of recognized authorities on these respective subjects; not so, however, with hydro-electric power propositions, which are most frequently begun in a hap-hazard sort of fashion, with the stream and a fall as assumed assets, while the market, constancy of output, cost of product, riparian rights, and numerous other controlling features remain undetermined until some later day. Hence promoters of hydro-electric projects have not found the investing public at all eager to take their securities, because of the general and well-grounded impression that their presentations are not entitled to the same degree

of confidence as other undertakings command, nor can there be any hope for a tide in their favor until such confidence be inspired.

Publicity of the realities of a subject will always carry conviction of merit, if such there be; and much of the reluctance of capital to recognize the indubitable value of investments in hydro-electric power plants is no doubt due to the paucity of the proper sort of educating literature on this subject.

This at least is the judgment of the author, born of the experience gained by some fifteen years of exclusive hydro-electric power practice, and this is the reason and purpose of this volume,—to place within reach of the promoter, investor, and practitioner an analytical treatment of hydro-electric practice in all its phases from inception to realization.

This subject is treated in two parts; the first is entitled "Analysis of a Hydro-electric Project," and is written for the layman, being devoid of technical treatment, and may therefore be characterized as the commercial essence of the subject. The author believes that an intelligent perusal of this first part will insure the reader against those errors of commission and omission which cause most of the failures in these projects. No engineering training or experience is required clearly to follow and fully to appreciate and understand the presentation of the analysis, which covers the topic as completely as can be done without the introduction of the technic.

CHAPTER I. treats of the market of electric current, where it may be found and how its value is readily determined. This is purely a commercial subject and ranks first in importance in the analysis.

CHAPTER II. discusses the power opportunity, how the available flow can be ascertained and the fall, and from these the power output on which the project may be safely based.

CHAPTER III. relates to the feasibility and practicability of the development. It treats of questions of riparian rights, Federal and State control of streams, economical limitations and of the investment balance.

CHAPTER IV. gives a non-technical synopsis of the cost of such a project, with general reference to the separate features of the required works and equipment; and

CHAPTER V., which closes the analysis, reviews the value of the project as an investment and suggests its proper presentation.

In this part are incorporated sixteen diagrams, from which the reciprocals of hydraulic, mechanical, and electrical power, horse-power and kilowatts, the flow over spillways and from reservoirs, fixed charges of and revenues from hydro-electric plants, and the approximate quantities for dams, foundations, power houses, embankments, bulkheads, and transmission lines may be readily ascertained. Other subjects, such as drainage areas, precipitation belts, and report and plans, are suitably illustrated.

PART II., "Designing and Equipping the Plant," is written for the student and practitioner. The arrangement is in the logical sequence of the pursuance of the plan. The aim of the author has been to render the treatment complete in all its phases, with the exception of presupposing a knowledge of the principles of surveying and the rudiments of hydraulics, hydrostatics, and dynamics.

Each subject involving static or dynamic principles is analyzed from its basic functions, and all formulas are developed in elementary progression; wherever practicable without complexity of methods or deductions, the useful constants are reduced to diagrammatical forms, which become available for ready reference in application. All features of importance are illustrated by sketches or views from existing plants, chiefly such as have been designed and constructed by the author, and the quantities of materials for all structures are given, for useful units, in tables or diagrams.

CHAPTER VI. treats of the surveys, embracing examination of maps, reconnaissance, topographic, stadia and photo-topographic operations, triangulation and levelling, flow measurements by different methods and deductions of run-off from precipitation and evaporation.

CHAPTER VII. deals with *development programmes*. This discussion covers the many possibilities presented by various conditions, with illustrated examples of the most important and frequent occurrences.

CHAPTER VIII. embraces *structural types*, beginning with definition of terms and methods, including the theory and constants of concrete-steel construction, methods of *coffering* preparatory to dam and power-house construction, with tables of quantities for dikes, cribs, sheet pile and wall curtains, and the various types of *cut-off* structures.

The treatment of *dams* and *spillways* is introduced by an exhaustive analysis of the basic theories of pressure and resistance and of all the underlying principles, with original determinations of practical constants

for a variety of designs, and their detailed parts, such as foundation, substructures and superstructures, and appurtenant features. The various phenomena influencing the design of dams, such as overflow, backswell, and the control of flood discharge, are fully analyzed. Especial attention has been given to this topic of dam designs; it is rudimentary from start to finish, with some original conclusions, it is believed.

The concrete-steel *gravity dam* design is fully detailed as to theory and practical execution; so are several types of the open spillway, flashboards, sluices, gates, fishways, and log chutes. Timber spillways are likewise treated, with stability discussion and quantity tables. The retaining wall theory is presented in connection with abutments, as are embankments, bulkheads, and reservoir structures of various forms.

Diversion works embrace open channels, flumes, pipe lines of concrete, steel plate and wood stave, with theories of flow, slope, and velocities fully analyzed and development of practical constants tabulated for all the different ranges entering this subject.

Power station follows, with all the practicable variations fully illustrated and described and with tabulated quantities. This subject is treated also in detail of foundation, pits, penstocks, and operating floor, and considerable space is devoted to their full description and to illustration of existing plants. *The submerged power station,* which represents the most recent developments in hydro-electric practice,—that is, the location of all power equipment in the interior of a hollow concrete-steel spillway,—is described and profusely illustrated by views of the first of this kind of plant recently completed.

Chapter IX. treats of the *power equipment,* with theory of turbine designs and efficiencies, dimensions and output constants, the latter being reduced to diagrams. This treatment of turbines has been compiled with especial care, for the purpose of conveying a clear conception of this most important topic of hydro-electric practice. Hydraulic governors are also described and illustrated.

Electric equipment follows, with a brief treatment of the magneto-electric theories, current symptoms, design and efficiencies of dynamos and their regulation. Dimensions and output of generators are reduced to diagrams. *Transmission* of electric current is introduced by an analytical theory of current transformation and conversion, determination of line capacity, and the designs of line supports, wire fastenings, and insulators.

Chapter X closes Part II with a brief generalization on the preparation of plans, estimates, specifications, and of the engineering control of constructions.

The author fully realizes that many features relating to hydroelectric practice are herein treated on the surface only, and he hopes to present them in exhaustive detail in a future work. This refers principally to the maintenance and operation of hydro-electric power plants and such detail subjects as the operation of the generating, transmission, and distributing plants, with chapters on underwashing of foundations, embankments, and retaining walls, and repair methods; flood rises, head fluctuations, anchor and slush ice formation and practical safeguards; trash-rack functions, gate operations, pipe-line defects, maintenance and repairs; turbine regulation and management; electric generating plant phenomena and their practical solution; transmission accidents and remedies; substation practice for lighting, industrial, and traction power and electric heating service; commercial rates and business management; valuations of various electric properties, such as light plants and railroads, and statistical facts of operating costs and earnings compiled from existing plants located in various sections and of different capacities.

The author has drawn freely upon standard works for the various topics covered by this subject, especially Hydraulics and Water Supply, by J. T. Fanning, C.E., Hydraulics, by M. Merriman, C.E., Hydraulics and Hydraulic Motors, by Julius Weisbach, Ph.D., Hydraulic Motors, by G. R. Bodmer, A.M., Electric Motors, by S. P. Thompson, D.S.C., and Electric Transmission, by Louis Bell, Ph.D.

In closing, the author wishes to give expression of his full appreciation of the services rendered by Mr. K. Asker, C.E., who prepared all the drawings.

H. A. E. C. von Schon.

Detroit, Mich., June, 1908.

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PART I

ANALYSIS OF A HYDRO-ELECTRIC PROJECT

When it is desired to examine an undeveloped water-power for the purpose of producing electric current as a commercial commodity, experience has taught that it is best to ascertain first where the product will find its market, and to follow up a satisfactory showing at that end by determining the power capacity of the source, the feasibility of harnessing the same, and the cost of accomplishing this. Observing this programme, which is really that adopted in any commercial enterprise, is found to insure a reliable conclusion.

CHAPTER I

THE MARKET

The market of electric current is to be found in any community. The commercial unit is the product of quantity of current and time of service; the measures are one electric horse-power, or one kilowatt, and the year and hour. The horse-power-year is the basis of mill and factory power contracts, while the kilowatt-hour is most generally adopted for lighting, shop, and traction service, because of the greater fluctuations from continuous operations.

Diagram 1 gives converted values of the horse-power and kilowatt.

Electric current finds its market for lighting, industrial, and traction service.

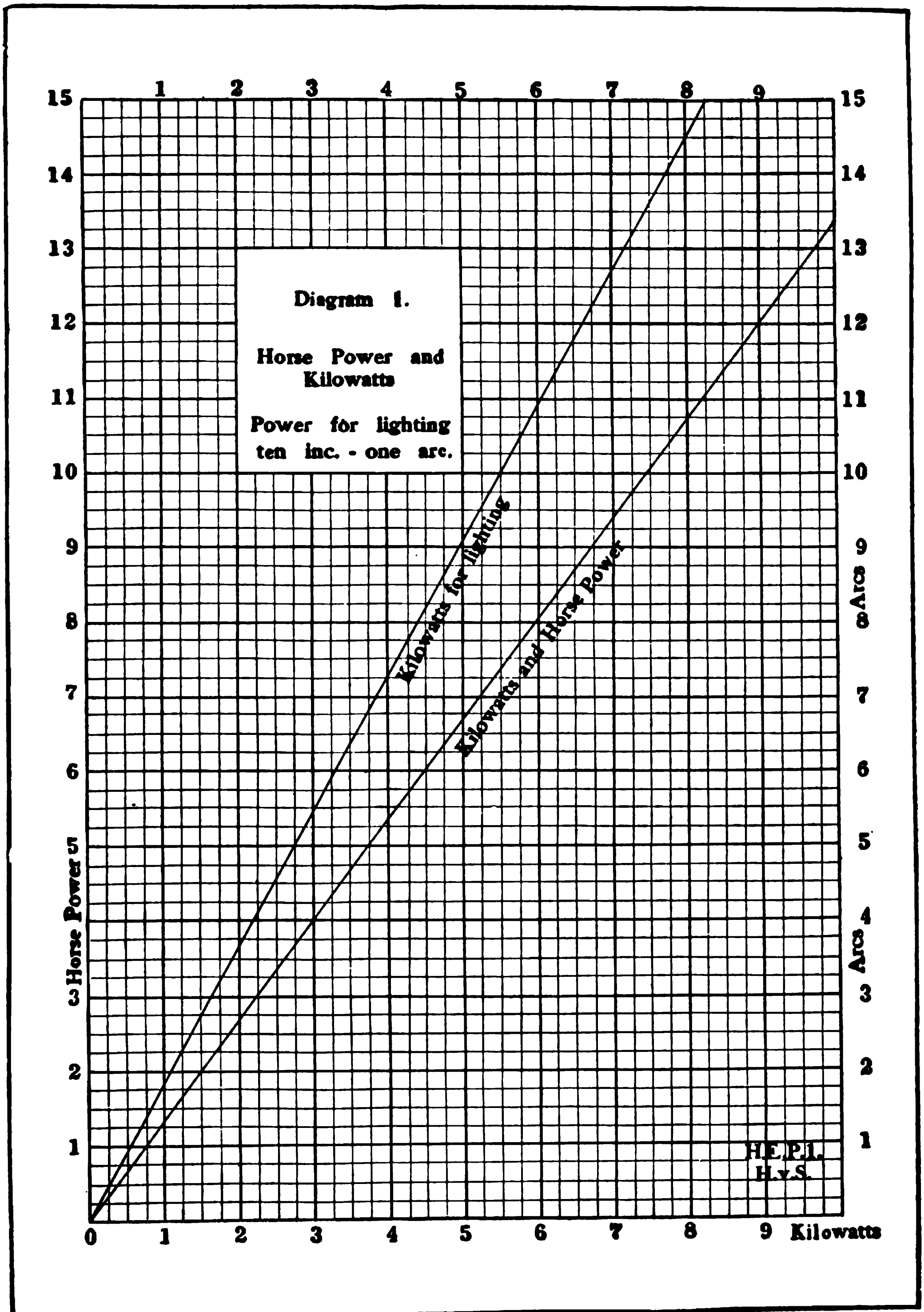
Article 1.—*Lighting* service consists of arc and incandescent lights. *Arc lights* are chiefly used for street, store, and hall illumination, and they generally require 550 watts. *Diagram* 1 shows relative current and power for any number of arcs.

Arc service is either of all-night or moonlight schedule, and the rates are most frequently per lamp per year, ranging from $50 up, the price depending solely upon local conditions. Occasionally arc-lighting service is per lamp-hour. This branch of current service is generally remunerative, and grows with the community if the service is at all satisfactory. Large stores and public halls, railroad depots, freight yards, and shipping docks are all light customers.

Incandescent lights are principally of 16-candle power, requiring 55 watts, or one-tenth of the arc-lamp current. *Diagram* 1 serves to find relative power. The rates of incandescent lighting are either per lamp per month, being denominated the "flat rate," or per kilowatt-hour, mostly of a sliding scale, the price lowering as total monthly consumption increases; this is termed the "meter rate." In cities incandescent-light service may be estimated at 700 hours per year per lamp, representing about 38 kilowatt-hours; in rural districts it is somewhat less. Meter rates range from 8 cents per kilowatt-hour up, depending entirely upon conditions of supply. The field for this business is in every dwelling, store, shop, mill, factory, and public institution and gathering place; the number of incandescent lamps which can be placed may generally be taken as equal to one-half the population.

ARTICLE 2.—*Industrial service* comprises the motive power used in shops, mills, and factories; the current is furnished as horse-power per year or month to industries operating for regular periods and with full loads, and on kilowatt-hour measure where periods and loads fluctuate. This service reaches every industry; for instance, the laundries and printing-shops, barbers, hotels, and all places using pumps, fans, and elevators, saw-mills and turning-shops, grist- and flouring-mills, machine-shops, textile, woollen, cotton, and knitting-mills, pulp and paper industries, wagon, buggy, automobile, furniture, piano, organ-factories, and, in fact, every class and kind of industry, none of which are too small to use power in some form or other. The power quantity used depends upon the character and size of the industry; a laundry may use 10 horse-power, printing-shops about 5 horse-power per press, wood-working shops 5 horse-power and metal shops 10 horse-power per machine, grist-mills one-half horse-power and flour-mills one-third horse-power per barrel capacity, ground wood-pulp mills 75 horse-power, sulphite-pulp 13 horse-power, paper-mills 18 horse-power per ton output.

ARTICLE 3.—There is also a *special class of manufactures* in which



For arc lamps of 550 and incandescents of 55 watts.

electricity, in its heating or decomposing capacity, forms the most important part and cost of the process, such as the production of calcium carbide, carbolite, carborundum, and glass; the group which requires such intense heat as 4000° to 7000° F., which can be obtained more economically from the electric current than from any other source; also the manufacture of soda ash, bleaching powders, and aluminum, which are produced by electro-chemical processes. These seek locations where electric current can be supplied cheaply, provided they have satisfactory transportation facilities. The proximity of the raw materials from which these products are manufactured is desirable, but secondary to the current supply; these raw materials are limestone free from clays and magnesium, anthracite or good coking coal, sand high in silica, salt, and clay high in aluminum. The amount of current required by them is large,—250 horse-power per ton of carbide or carbolite, 450 horse-power per ton of carborundum, and 1400 horse-power per ton of aluminum.

ARTICLE 4.—*Traction* is the third class of current service, and concerns street and suburban railroad systems. A street car is generally equipped with two 25 horse-power, an interurban car with four 50 horse-power motors; but the full amount of this power is required only for the starting of a loaded car on an up grade, while a going car on the level requires but from 10 to 50 horse-power, depending upon the speed; the amount of power current used, therefore, fluctuates greatly and constantly. The service period covers from 16 to 18 hours. This service is supplied by a variety of schedules, from the horse-power-year to the kilowatt-hour, with more or less supplemental conditions regarding maximum and minimum supply; the rates are from three-fourths to one and one-half cent per kilowatt-hour.

ARTICLE 5.—*The highest remunerative value* of current service is found in ten-hour (day service) industrial power of the smaller units but steady loads, together with the arc and incandescent lighting; such a service combination practically doubles the output,—that is, each horse-power performs double service, with some provision for the lapping of the two separate duties for a short period in the evening. It is not uncommon that the earning per horse-power with such a market will be between $50 and $60 per year.

ARTICLE 6. *Analysis of the Current Market.*— For the purpose of furnishing data to be considered in connection with weighing the commercial possibilities of a hydro-electric project, something better than

the fact that the development is within a few miles of a certain city where more steam-power is being used than this hydro-electric project can develop, is required.

This commodity, power, must, like all others, find a market on commercial principles, the essence of which is "*equal quality and service at competitive cost,*" and it is therefore necessary to collect data relating to the service and the cost in order to analyze the market possibilities.

A list of all existing power plants of any city, their condition, output, operating personnel, fuel consumption, and the periods and character of power service furnished by them can readily be obtained from insurance agents, the fire department, boiler inspectors, and assessors; the same information concerning central plants may be had from personal inspection, and the rates paid for power and light current service from the consumers. A complete power canvass of this sort should form the basis of the market investigation, from which it may be made more or less clear how much of this power business can be secured for hydro-electric service.

If the prospective customers are to be found among the present power consumers they must be convinced of the advantages of the electric as compared with the mechanical drives in shops and mills because of the increased power transmission efficiency, independence of the machine arrangements from the location of shafting, comparative noiselessness and cleanliness, and greater practicability for machine speeding. But the chief influence must come from the cost. The customer will have to make some investment in motor equipment, and he must therefore be convinced that it will be advisable to do so; that he will pay only for as much power as is doing actual work for him, which will be a reduction of ten per cent. and more from his present necessary engine output; that the cost of his power will be in future independent of labor or fuel prices, boiler, pump, or engine breakdowns and repairs; that there will be absolutely no power expense from the moment the machine stops until it starts up again; that the maintenance of his future electric drives will be nil as compared with that of his present mechanical drives; that his insurance will be much lower; and, last, but not least, that he can make of his power charge a fixed item, while now it is to him practically a speculative one.

The question, what does his power cost him, will bring him frequently to the realization that he cannot tell. Wages, fuel, oil waste and water cost are likely enough known; but what about renewals, repairs, and

depreciation of plant? If the shop tools require, say, 50 horse-power, he must have a 75 horse-power engine and 100 horse-power boiler; the fuel consumption will be not less than five pounds per horse-power hour; the full steam head must be on whether one or all of the tools are running, and it will be a quite frequent condition that a 5 horse-power drill, the other shop tools being idle at the time, is being operated with a 100 horse-power boiler output, the excess of course being blown off or wasted. What is the cost of the power output per horse-power during such a condition? What would it be if the drill were operated by an individual electric motor? This sort of reasoning will convince the steam-power user that the investment in electric drives will prove a money maker.

The central steam-electric plant cannot compete against hydro-electric output from a normally conditioned plant, no matter what fuel it burns or what its output efficiency. Those of steam-turbine equipment in five or ten thousand kilowatt units may produce current of continuous output at half a cent per kilowatt-hour, representing all charges excepting depreciation of plant, this is the very lowest yet reached and represents a 22 hour mill day, $40.15 per kilowatt year, which revenue would pay all charges and leave a considerable surplus on hydro-electric current which is developed within a total cost of $150.00 per kilowatt delivered, and this cost is representative of normal conditions of development and transmission. But the central steam plant's output runs more frequently above one cent per kilowatt-hour than below; it increases with fuel cost, small units, and inefficiency of equipment, none of which causes exist or are likely to occur with hydro-electric plants where the everflowing stream is the fuel, the size of units is indifferent, while equipment is likely to be modern and its efficiency is independent of the care and attention of operatives. The former statement, therefore, that a central steam-electric plant cannot compete with a normally conditioned hydro-electric plant, is absolutely correct, and it becomes only a question of cost and tact to secure this class of business for the hydro-electric project. In the majority of cases it will be found advisable to purchase the existing steam-electric central plant and merge it with the hydro-electric project, thus giving to it the status of a going concern, which is more advantageously and readily financed than an all paper proposition, so called.

Market investigations carried on in this manner will make it quite clear what volume and class of current business may be counted upon for

the hydro-electric output when ready to be delivered. They will certainly form a conservative basis for an estimate of first revenue, and none other should be taken. When the current is on the market at rates which are no higher than steam-power or other illuminants, only ordinary business management will be required to sell it wherever power or light have been used before, while increased consumption will come with the growth of the community and of its industrial interests, which, however, may be wonderfully accelerated by the liberal policy of the purveyors of electric current.

Of the cost of steam-power current and rates for hydro-electric service some data will be found in Part III., which treats of "Operating and Maintaining the Plant."

CHAPTER II

POWER OPPORTUNITY

WATER-POWER is the physical effect of the weight of falling water. One cubic foot of water weighs 62.5 pounds, and when this mass falls one foot the resultant energy is 62.5 foot-pounds, when it falls ten feet it is 625 foot-pounds.

The unit of power is the energy which lifts 550 pounds one foot in one second, being termed one horse-power, and water-power is therefore expressed by the product of the weight of water and height of its fall divided by 550, the period of time being one second,—*i.e.*,

$$\text{H. P.} = Q \text{ (flow per second)} \times 62.5 \times h \text{ (fall)} \div 550.$$

The components of the power product are, consequently, flow and fall.

The horse-power equivalent of the product of a hydro-electric development is *electric horse-power*, being so much of the original water-power energy as remains available for work in the shape of electric current.

ARTICLE 7.—*The flow* is the volume which passes a given point continuously.

Precipitation is the prime source of stream flow; it is the only source, excepting of rivers in the Arctic region which are yet glacier fed.

Rain, dew, snow, hail—all these are embraced under *precipitation*.

When rain falls upon the ground, its greater part sinks into the soil; the other portion runs off the surface into the catchment basin, or remains for a time on the surface if it be level. The quantity which passes into the ground depends upon the conformation of its surface, the porosity, depth, and degree of saturation of the material comprising it; this is the *ground water*; the portion flowing off the surface is the *storm run-off*.

Ground water continues to sink down and to move laterally by force of gravity as rapidly as the voids and frictional resistances of the material through which it passes permit, and finally finds its way to its destination, the nearest stream valley; but part of the ground water is retained by the roots of vegetation, rises through them up the stems and tree trunks, is converted into cellular fibre or exhaled as vapor from

vegetation, or from the surface, into the atmosphere, to be collected and again precipitated.

Precipitation falling on water partially flows toward the destination and part of it is vaporized.

All the water which is consumed by vegetation and which is vaporized is *evaporation;* all that portion which finds its way into the stream is the *run-off*.

As precipitation is the source, so is ground water the sustenance of stream flow, and the storage capacity of the ground is the key by which the flow characteristics must be sought; in other words, while precipitation is essential as the source of flow, its quantity is not necessarily an index of proportionate flow; ground storage and the constancy with which it feeds the stream are the criteria to be applied when stream flow is studied.

It becomes first necessary to measure the drainage area and examine its topography, geology, flora, and culture.

ARTICLE 8.—*The drainage area* comprises all that country which drains into the system of the stream above the point under investigation.

A topographical map shows the *water-shed*, height of land, or divide along which the drainage separates, passing down in opposite directions into the feeders of adjacent streams, and its demarcation will give the correct boundary of the drainage area. Topographical maps are not always procurable, nor do such exist excepting for limited areas to which the operations of Federal or State surveys have been extended. When such a map is not available, the drainage area may be found from State or County maps by marking its boundary as passing midway between the apparent heads of tributaries or parallel watercourses draining into neighboring stream systems. This method will result in a very close approximation of the correct drainage area. The unit measure to be applied to the marked area is the square mile as per scale of the map utilized. Care must be taken in this operation, especially when the map scale, as is frequently the case, is fractional; when enlargement is made by pantagraph, the ratio must be verified from a standard unit and multiples of it. State and County maps of Northern, Western, and Middle States show the section and township lines, marking the areas of one and of thirty-six square miles, respectively. This is not the case in the Eastern and Southern States,—that is, in the territory of the original colonies,—in which geographical subdivision boundaries pay no heed to meridians

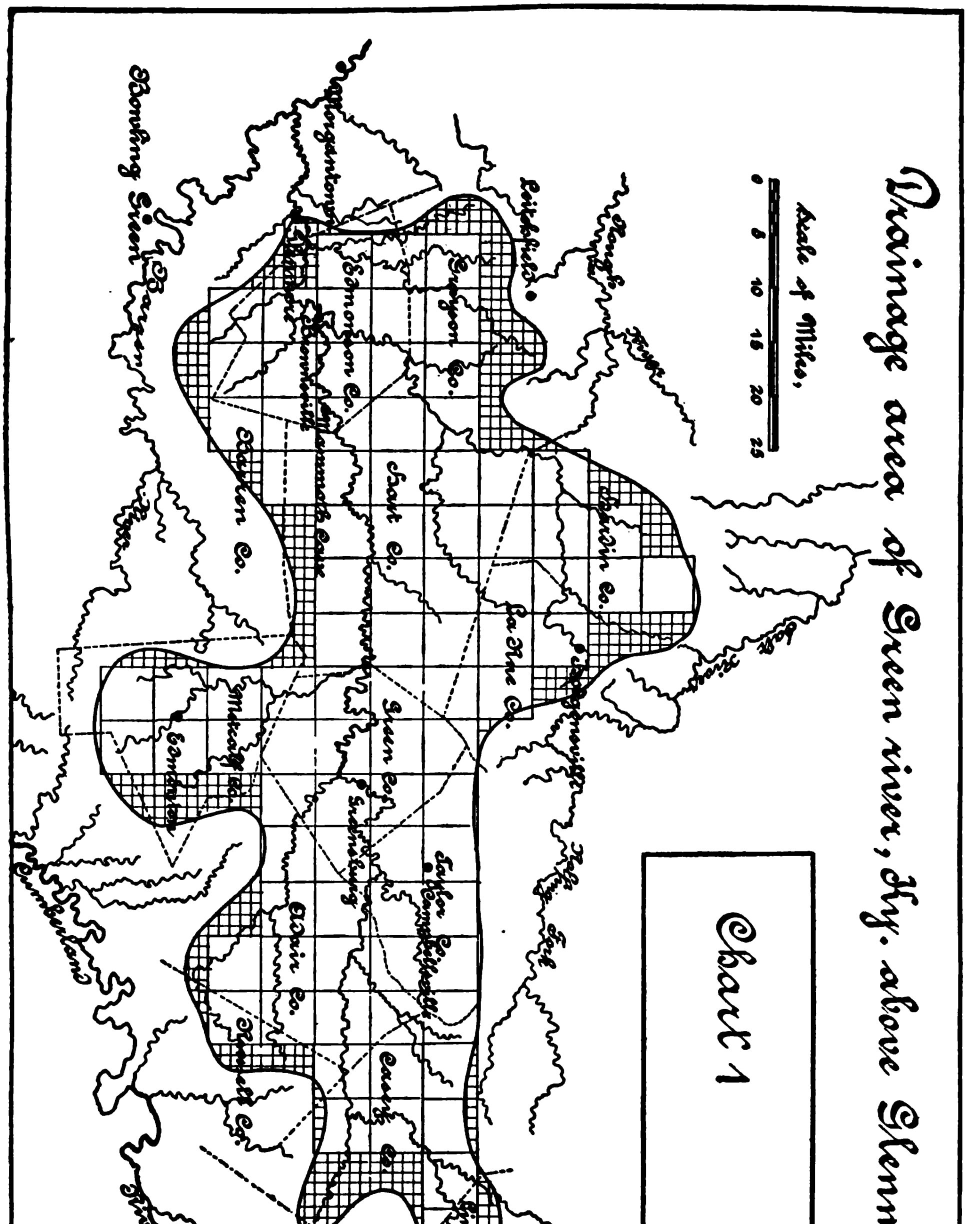
and parallels. When topographical maps are not available County or State maps furnish the only material from which drainage areas can be taken with reasonable accuracy; the streams are usually located correctly on these.

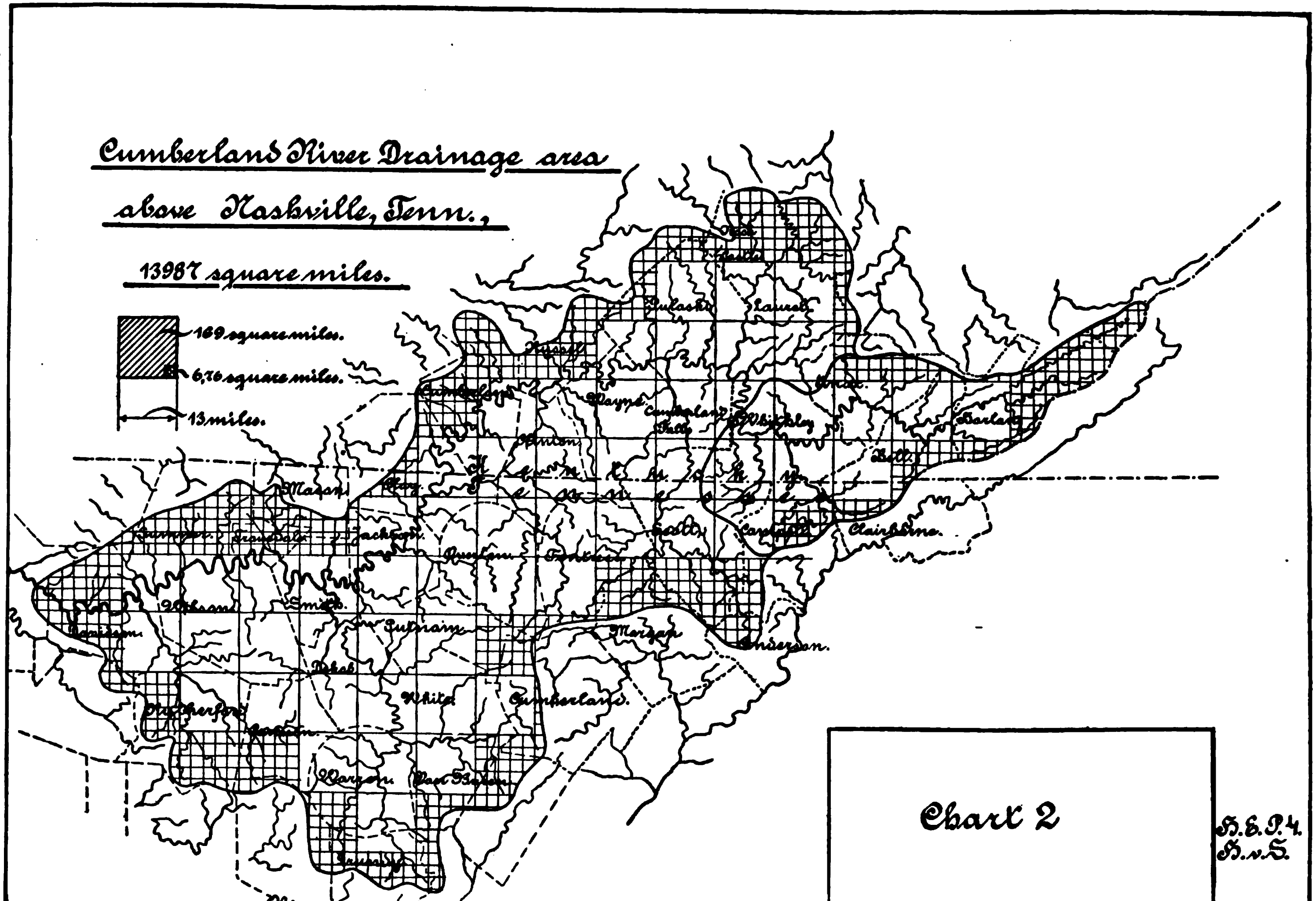
Chart 1 shows the drainage area of Green River, Ky., above U. S. Lock No. 5. It was taken from a State map, as the U. S. Geological Survey had not then covered that section. Frequently the drainage area for a certain point can be closely estimated by reference to the area of points up or down stream on the same river, for which the areas are given in the U. S. Geological Survey records of the stream measurements, which are published annually, and which now cover the majority of important rivers in the United States. Generally speaking, large drainage areas can be taken from wall maps of the United States and of Canada and those of small areas from State and County maps.

However it must not be inferred that this matter of drainage area and its correct determination is of small importance, on the contrary it forms one of the principal fundamental factors from which the flow of the stream must often be found and if considerably in error, especially in considering small watersheds, the mistake may be a costly one.

Article 9.—*The topography* of the drainage area represents the undulations of the surface, which exercise an important influence upon the distribution of run-off. It is self-evident that the storm run-off will be greater from a hilly country than from table-lands and that the part of precipitation remaining available for ground storage will be correspondingly smaller; the topography, therefore, is one of the fundamental conditions determining constancy, fluctuations, and flood characteristics of stream flow and should receive commensurate consideration.

A topographical map of the United States is published by the United States Geological Survey, from which the general topography of all important drainage areas can be readily found,—that is, it can be learned whether the area is generally of a hilly or flat country, whether stream channels are narrow and declivitous or broad valleys, and what, if any, is the area of lakes and swamps. It is not necessary to define these conditions as to details, but it is essential to understand the prevailing features as distinguished between the flat land drainage areas of streams in Iowa, Illinois, Indiana, and the middle West, and those of


Drainage area of Green river, Ky. above Glenmore.
Scale of Miles.
0 5 10 15 20 25
Chart 1
Grayson Co.
Edmonson Co.
Hart Co.
Barren Co.
Green Co.
Metcalfe Co.
Adair Co.
Casey Co.
Taylor Co.
Russell Co.
Green River
Cumberland


Cumberland River Drainage area
above Nashville, Tenn.,
13987 square miles.
169 square miles.
6,76 square miles.
13 miles.
Pulaski
Laurel
Russell
Wayne
Cumberland
Clinton
Whitley
Knox
Harlan
Bell
Claiborne
Campbell
Scott
Fentress
Overton
Jackson
Clay
Macon
Sumner
Trousdale
Wilson
Smith
Putnam
Morgan
Anderson
Dekalb
White
Cumberland.
Rutherford
Warren
Van Buren
Chart 2

rolling and hilly formation in Virginia, Kentucky, Vermont, or the Pacific slope; or of parts of a river, when perhaps a site on its upper reach is considered, where practically all of the area is in mountain ranges or foot-hills, or a point near the mouth of the river, where only a small part of the area is in hilly and by far the greater is in flat or rolling country.

As an example, the Cumberland River may be cited, of which Chart 2 gives drainage areas for two important water-power sites,—the upper one at Cumberland Falls, Ky., often called the Niagara of the South, the lower one near Nashville, Tenn.; the first is almost wholly in the mountain region, while the Nashville site area is largely of flat and rolling country.

Such general information, as has been said before, can be gleaned from the United States Geological Map or from the Map of Altitudes published by the same department. Some conclusions as to topography can frequently be drawn from location of railroads and highways, especially from the former, which in hilly and broken country more generally parallel the watercourses than in flat or rolling parts. Many sections have been covered by detail topographical surveys, and the information then is conclusive.

To secure a sufficient appreciation of the topography from any or all of these sources requires considerable experience in the reading of the projections. It is, of course, prohibitive in cost to make surveys for this purpose, but a horseback reconnoissance or drifting down the river, or the major portion of it, will frequently enable the investigator to form correct conclusions on this point. The author has examined several rivers by going down in a canoe, and the information thus gained proved exceedingly valuable in planning power developments.

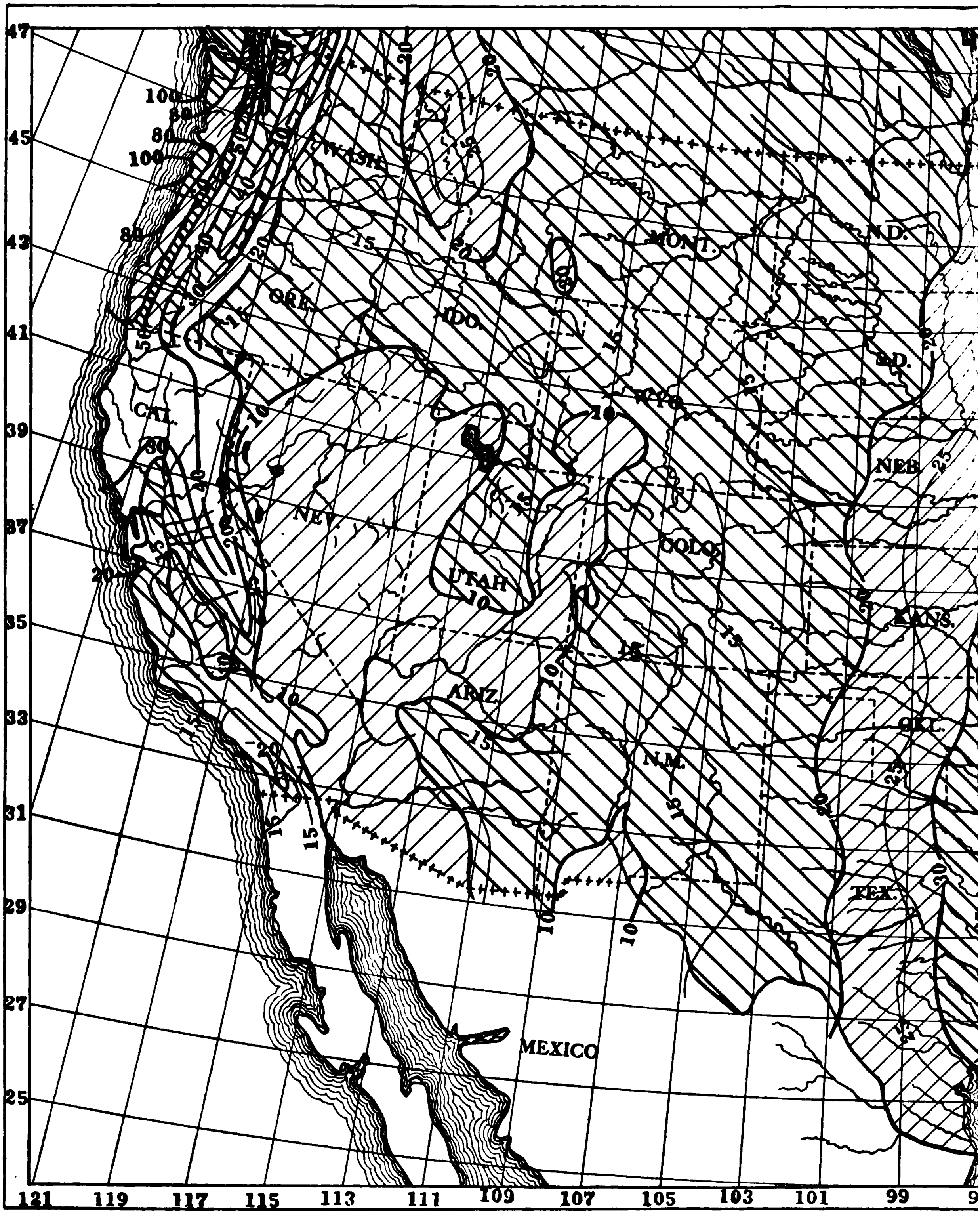
ARTICLE 10.—*The geology* of the drainage area should be understood to the extent of influencing the degree of absorption and storm run-off. Where rock ledge is at surface or crops out in banks, the depth of overlying drift is readily ascertained; otherwise borings should be made, if practicable to rock, and the character of the overlying material found. The percolosity of the ground determines its storage capacity. A very satisfactory conception of this latter can be obtained from the observance of conditions following heavy rainfalls, when frequent accumulations of pools of water in level fields are evidence of the prevalence of non-absorbent soil, while its rapid disappearance indicates sandy and gravelly for-

mations. The stream itself gives testimony of these conditions, the water being turbid where draining clayey soils, while the run-off from porous earths shows little discoloration. No practical information, for the investigation in hand, can be gained from this study beyond the formation of the immediate subsurface, and this can be obtained by a personal examination of the locality in question and a study of the farming culture. In many sections conclusive data can be obtained from records of well borings, which are generally sunk by one concern covering several counties. Some of the State Geological departments publish such records.

ARTICLE 11.—*The flora and culture* in the drainage area constitute the third characteristic which influences the flow. Forests are conservers of water, protecting it from the heat of the sun and the winds, and thus retarding its evaporation as compared with cultivated fields, grazing land, or open soil, while the many obstructions to the storm run-off in timbered areas result in a greater portion finding its way into ground storage. The requirement of moisture for tree growth is considerably less than that of crops; for instance, long grass consumes six times as much water as fir trees. Wooded hill-sides, tamarack and cypress swamps are storage reservoirs; highly cultivated table-lands with tile drainage leave but little, if any, surplus during the growing season for run-off. It is important, therefore, to secure an adequate knowledge of these conditions. The degree of cultivation can be ascertained from the rural population of counties and the character and volume of produce shipped out, all of which, together with the forest area, can be obtained from the United States census publications.

ARTICLE 12.—As stated before, *precipitation* is the source of all stream flow, which latter can only be a fraction of it. Frequently a good enough preliminary estimate of flow can be made if quantity of precipitation in drainage area and the extent of the latter which contributes to the river at the point under examination are known, and reliable data of fluctuations of flow throughout seasons and years can be found from such information.

Drainage areas, as has been seen, can be readily measured, and precipitation is ascertainable from public records. For fifty years and longer rain- and snow-fall have been measured in the United States and Canada through the agency of the Government, in this country by the United States Weather Bureau, in the Dominion of Canada by Provincial meteorological departments; points of observations are distributed over



WASH.
ORE.
IDO.
MONT.
N.D.
S.D.
WYO.
NEB.
CAL.
NEV.
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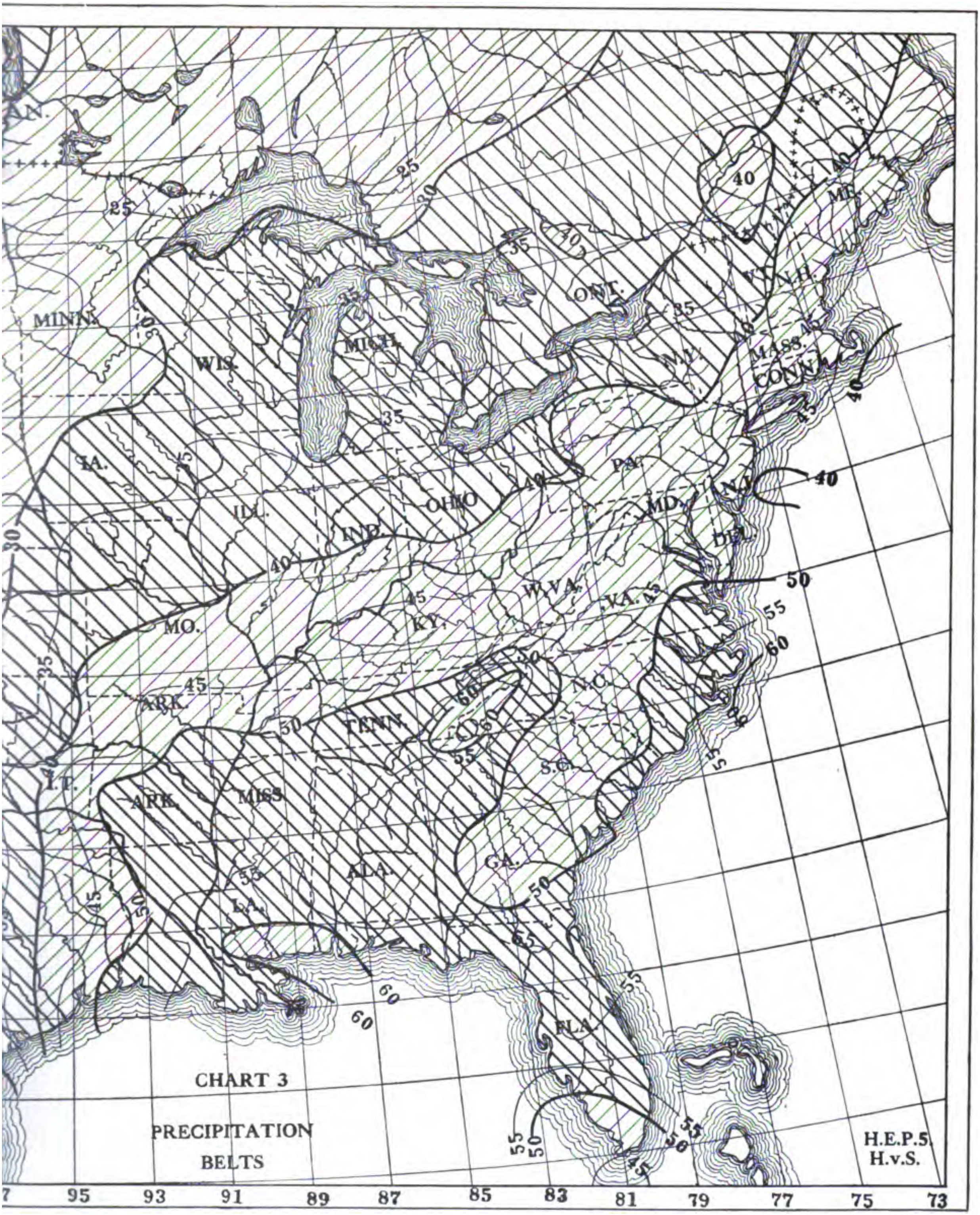
CHART 3
PRECIPITATION
BELTS
H.E.P.5
H.v.S.
MINN.
WIS.
MICH.
IA.
ILL.
IND.
OHIO
MO.
KY.
W.VA.
VA.
PA.
MD.
N.J.
DEL.
N.Y.
ONT.
MASS.
CONN.
N.H.
ME.
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the country with somewhat of a uniformity, and one or more of them can always be found in a certain drainage system. The measurements are made by means of standard cups, so exposed that they receive the normal rain- and snow-fall, which is measured as to its depth in inches and fractions, the snow being melted for that purpose and therefore expressed in the same quantity as the rain. Observations are made daily of the rain- or snow-fall during twenty-four hours, and the daily, monthly, and annual totals are given in public records.

Such measurements require no skill and may, therefore, be accepted as a fairly accurate record of precipitation.

From these data the general distribution of precipitation is well known; it is illustrated on Chart 3, on which equi-precipitation curves are projected, the precipitation being the normal annual quantity in inches. This may be used with advantage for first investigations of stream flow, exhibiting neither the wet nor dry but the normal year.

Observations of this character have been carried on for a sufficiently continuous period to warrant the conclusion that precipitation is not undergoing any great changes: it does not rain more or less now than it did fifty years ago, nor does the clearing of land seem to be followed by any marked change in rainfall in that section. The ordinary fluctuations of precipitation appear to be represented in a cycle of seven years,—that is, the totals of seven years of precipitation are very nearly equal; each of these seven-year periods contains one dry year, the one of least precipitation, and generally one or two extremely wet years.

Article 13.—The information to be sought in connection with *determination of stream flow* comprises the quantity and distribution of rain and snow during at least one complete cycle of seven years, in order to fix upon the ordinary dry year of the period. The safe method is to collect the precipitation data by monthly totals from as many observation points as are obtainable in the drainage area for a period of fifteen continuous years, which are certain to contain a complete cycle. If the entire drainage area is located in the same precipitation belt as per Chart 3, the monthly means of all observations are compiled and may be taken as applying to the entire area; when the drainage area extends through different precipitation belts, the monthly means of stations in each precipitation belt are to be found, the drainage area is to be divided into parts covered by different precipitation belts, and the respective monthly means applied to each. Such precipitation records from five points in

the drainage area outlined on Chart 1 served the purpose satisfactorily. Profile 1, page 17, exhibits the total annual precipitation during a period of fifteen years, the yearly fluctuations, the wet and dry years, and their periodic occurrence. The extremes are 35½ inches for the low and 53½ inches for the high annual precipitation; the mean of the fifteen years is 45.6 inches, and reference to the precipitation, Chart 3, verifies this as the normal precipitation for the State of Kentucky. The following is a summary of these annual precipitation conditions during the period of fifteen years:

ANNUAL PRECIPITATION SUMMARY.

Dry Years of less than 40 in.		Wet Years of more than 50 in.		Normal years 40 to 50 in.	
1894	3	1891	3	1892	9
1901		1893		1895	
1904		1898		1896	
				1897	
				1899	
				1900	
				1902	
				1903	
				1905	

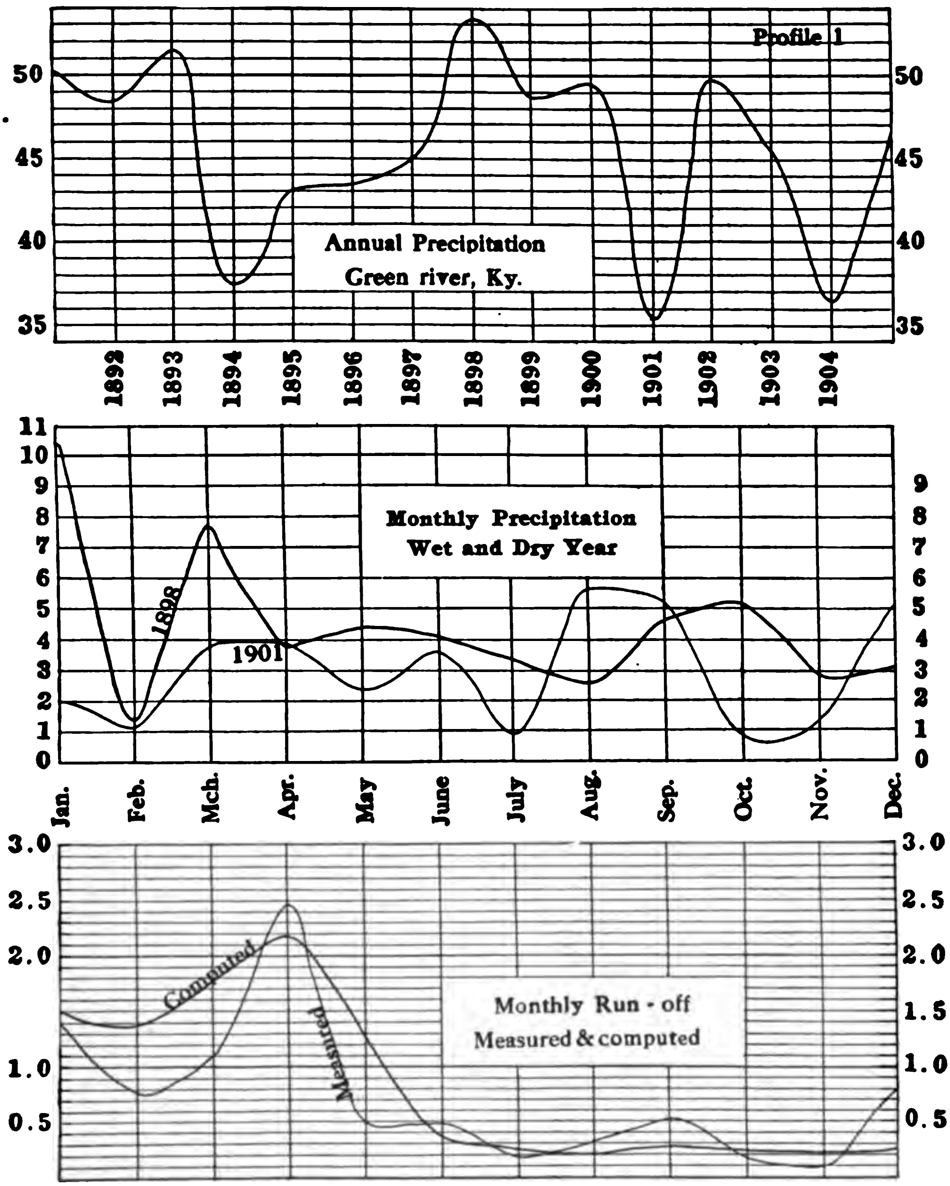
Profile 2, page 17, shows the monthly distribution of precipitation during the dry and the wet year; we are chiefly concerned now with the first of these—the year 1901, with a total of 35½ inches, ten less than the normal; the low monthly is 0.75, the high 5.75, and the mean 3 inches.

MONTHLY PRECIPITATION SUMMARY.

Low Months of less than 2 in.		High Months of more than 4 in.		Normal Months 2 to 4 in.	
February	4	August	3	January	5
July		September		March	
October		December		April	
November				May	
				June	

From these precipitation summaries all the flow supply conditions can be correctly learned.

ARTICLE 14. *Evaporation.* Investigations have been carried on in this country and abroad for many years with the object of finding a standard for evaporation ratio under the various conditions of precipitation, temperature and air currents, applicable to different surface formations and the moisture requirements of vegetation during its stages of rest and growth. Much research has been given this matter in Sweden and Germany and of late years in this country to evolve such a


Profile 1
Annual Precipitation
Green river, Ky.
50
45
40
35
1892
1893
1894
1895
1896
1897
1898
1899
1900
1901
1902
1903
1904
Monthly Precipitation
Wet and Dry Year
1898
1901
Jan.
Feb.
Mch.
Apr.
May
June
July
Aug.
Sep.
Oct.
Nov.
Dec.
Monthly Run - off
Measured & computed
Computed
Measured
3.0
2.5
2.0
1.5
1.0
0.5

H.E.P.6.
H.v.S.

practical method for evaporation determination, and observations and measurements of evaporation from water surfaces, forests, uncultivated lands, and fields with growing crops of all kinds have conclusively established such a ratio. Evaporation from water surfaces is found by aid of evaporation pans which are of considerable area and water-tight, the contents of which may be found precisely at known intervals of time, the diminution being due to evaporation. For this determination from land, known areas are isolated, surrounded by ditches in which the water draining off can be accurately measured, when the difference between the quantity of a certain rainfall and that draining off by these ditches represents evaporation from that area for a given time, other climatological conditions being duly observed. These experiments have been repeated in various latitudes by different persons, and from them certain evaporation values have been determined.

Table 1 gives monthly evaporation in inches from different surfaces for certain locations.

TABLE 1 *—EVAPORATION FROM WATER AT EMDRUP, DENMARK.

(Latitude, 55° 41′ N.; longitude, 12° 34′ E. from Greenwich.)

Year.	Jan. in.	Feb. in.	Mch. in.	Apl. in.	May. in.	June. in.	July. in.	Aug. in.	Sept. in.	Oct. in.	Nov. in.	Dec. in.	Total. in.
1849	1.1	0.3	1.8	2.5	4.1	5.8	4.7	4.0	2.6	1.1	0.9	0.6	29.5
1850	1.1	0.3	1.2	1.7	4.5	5.6	4.8	4.8	2.4	1.6	0.9	0.2	29.1
1851	0.5	0.4	0.7	1.7	4.2	4.8	5.7	5.1	2.7	1.5	0.6	0.5	28.4
1852	0.7	0.5	0.8	2.4	3.8	4.6	6.4	4.5	2.7	1.7	0.8	0.5	29.4
1853	0.5	0.1	0.7	1.0	4.1	6.2	5.1	4.2	2.8	1.1	0.6	0.5	26.9
1854	0.5	0.9	0.9	3.2	3.3	4.5	5.2	4.3	2.6	1.2	0.7	0.6	27.9
1855	1.0	1.1	0.5	1.2	2.6	4.1	4.7	4.1	2.8	1.4	0.9	0.7	25.1
1856	0.5	0.5	1.2	2.1	2.8	4.6	4.3	4.0	2.0	1.9	0.6	0.5	24.0
1857	0.7	0.6	0.6	1.4	4.1	6.6	5.9	4.3	3.2	1.4	0.7	0.4	29.9
1858	0.4	0.7	1.2	3.1	5.1	6.1	4.9	5.6	2.8	1.6	0.7	0.4	30.6
1859	0.3	0.5	0.7	1.9	4.3	5.8	5.3	3.8	1.8	1.0	0.7	0.3	26.4
Mean	0.7	0.5	0.9	2.0	3.7	5.4	5.2	4.4	2.6	1.3	0.7	0.5	27.9
Ratio	.301	.215	.387	.860	1.592	2.323	2.237	1.892	1.118	.559	.301	.215	
Mean Evaporation from Short Grass, 1852 to 1859, inclusive.													
Mean	0.7	0.8	1.2	2.6	4.1	5.5	5.2	4.7	2.8	1.3	0.7	0.5	30.1
Mean Evaporation from Long Grass, 1849 to 1856, inclusive.													
Mean	0.9	0.6	1.4	2.6	4.7	6.7	9.3	7.9	5.2	2.9	1.3	0.5	44.0
Mean Rainfall at same Station, 1848 to 1859, inclusive.													
Mean	1.5	1.7	1.0	1.6	1.5	2.2	2.4	2.4	2.0	2.3	1.8	1.5	21.9

* J. T. Fanning, Water-Supply Engineering.

TABLE 2.—EVAPORATION FROM WATER-SURFACE AT BOSTON, MASS, IN INCHES—FOURTEEN YEARS,*—1875–1890.

	1876.	1877.	1878.	1879.	1880.	'81–'84.	1885.	1886.	1887.	1888.	1889.	Total.	Mean.
January	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	0.96	15.36	0.96
February	1.05	1.05	1.05	0.15	1.05	1.05	1.05	1.05	1.05	1.05	1.05	16.80	1.05
March	1.70	1.70	1.70	1.70	1.70	1.70	1.70	1.70	1.70	1.70	1.70	27.20	1.70
April	2.98	2.98	2.98	2.98	2.98	2.98	2.98	3.12	3.07	2.78	2.84	47.57	2.97
May	4.45	4.05	4.14	5.89	5.22	4.45	3.77	4.45	4.83	3.35	4.57	71.42	4.46
June	5.44	5.68	5.26	5.32	6.46	5.55	7.01	5.25	5.05	5.98	3.94	88.69	5.54
July	7.50	4.82	6.04	6.41	5.82	5.98	7.09	5.59	5.96	5.57	5.04	95.72	5.98
August	6.21	4.40	4.33	5.23	5.34	5.50	7.41	5.80	6.20	5.81	4.25	87.98	5.50
September	3.48	4.08	4.04	3.80	4.04	4.20	5.13	4.55	4.57	3.91	3.08	65.88	4.12
October	3.12	2.51	3.52	2.99	2.79	3.11	2.79	4.13	3.61	3.27	3.13	50.52	3.46
November	0.66	2.23	2.23	2.23	2.60	2.23	2.23	2.69	3.00	2.71	1.98	35.94	2.25
December	1.51	1.51	1.51	1.51	1.51	1.51	1.51	1.51	1.51	1.51	1.51	24.16	1.51
Total	39.06	35.97	37.76	40.07	40.47	39.22	43.63	40.80	41.51	38.60	34.05	627.24	39.20

Evaporation from an entire drainage area of a stream is readily determined if precipitation and flow for a sufficiently continuous period—as, for instance, one year—are known. Precipitation may be measured as already described, while the actual quantity of water passing down the stream may also be found by physical measurements. This has been done on many streams for long periods, measuring weirs have been erected and the overflow recorded by automatic gauges, and thus the continuous and total flow per day, month, or year determined; the difference between this flow and the total precipitation is chargeable to evaporation.

From such data, resulting from extensive systematic investigations, rules have been evolved to find evaporation, the correct application of which will give results in harmonious agreement with those found by measurements, and these rules or formulæ may be applied to any set of conditions for the purpose of finding similar results.

A detailed description of the method of determining evaporation will be found in Part II.; a practical application of it, from the author's practice, will go far toward securing for its further study that degree of confidence in its value for the general investigation of stream flow which it deserves.

Green River, Ky., was the subject, the development of power to be planned at a Government lock. The drainage area was delineated from a State map, as shown on Chart 1; precipitation data were collected as

* Desmond Fitzgerald, C.E., Rainfall, Flow of Streams, and Storage. (Transactions Am. Soc. C. E., Vol. XVII.)

given on Table 3 and on Profiles 1 and 2; from these evaporation and run-off were computed, the results for the latter being plotted on Profile 3, in heavy line, as monthly run-off for the dry year of the period covered by precipitation records.

The overflow at the Government dam had been recorded daily for this same year, and from these the flow was computed from the weir formula, and these results are also plotted on profile in light line. Both of these operations were executed by different persons. The agreement between the two profiles is striking, the discrepancies during the winter months are accounted for by the fact that the run-off as computed was partially frozen and therefore retained until spring. The reliability of the computation method is, however, evident. Similar results can be quoted from published records, especially in New England and Eastern States, where the flow of many streams has been measured in similar manner for many years and the run-off as computed from evaporation compared with it. In fact, the system has been evolved from comparative results on rivers where flow has been established by physical measurements. The author has always used the two methods of calculation and measurements to check results whenever authentic flow measurements covering sufficient periods were available, and has found them to agree most satisfactorily.

ARTICLE 15.—*The application of flow deductions from precipitation* for preliminary investigation purposes is expressed by the following rules:

Rule 1. About 30 per cent. of annual precipitation remains available for flow; the other portion is evaporation.

Rule 2. The low monthly flow cannot exceed one-twelfth of the total available flow.

Rule 3. The monthly flow during three months, generally in the fall, in Northern latitudes is from one-half to one-third, in Southern latitudes from one-fourth to one-sixth, and in Western States from one-sixth to one-tenth of one-twelfth of the annual precipitation excess over evaporation.

The author has met cases where the application of this simple rule would have saved to the promoters of water-power projects thousands of dollars. One in point is recalled, where a water-power was projected on a stream with a drainage area of about 700 square miles, the available fall was 30 feet, and the opportunity was credited with a power output of 3500 horse-power, which would call for an available flow of

about 1450 cubic second feet, requiring run-off of 2.07 cubic second feet per square mile of area, which represents monthly precipitation excess over evaporation of 2.3 inches. These facts should be sufficient to reveal a gross error in the assumed power output,—that is, if the area has been ascertained. The stream was in a Northern State where normal annual precipitation is 35 inches, annual temperature about 48°, and annual evaporation therefore nearly the same as that found in the example given, or about seven-tenths, leaving a residue of precipitation for annual run-off of about three-tenths, or eleven inches for the entire year. As a matter of fact, the opportunity was good for about 1500 horse-power with a 250 horse-power auxiliary plant supplementing the three months low flow output. About 1000 acres of lands had been purchased and paid for, but no power development has yet taken place. Many similar instances could be cited.

ARTICLE 16.—*Flow measurements* are made by various methods, but, unless they extend over a sufficiently long period, especially covering the low stages, their result is not conclusive as to the available flow. As a rule, the time necessary to do this properly cannot be taken; when water-power projects ripen to the stage of such investigations, results are expected promptly, and, unless it is then the period of low flow, measurements will be of little practical value. On many streams measurements have now been made for several years by the Federal Government, the results being published in annual reports of United States Geological Survey as stream measurements, and, where these have been carried on long enough to furnish a rating table for the stream, this information may be taken as conclusive for the purpose provided always that it is correctly applied to the purpose it is intended to serve.

The most reliable method of measuring a stream's flow is by an overflow weir, which is a type of low dam over which the entire flow passes. It is, of course, necessary that no portion of the flow passes under it or around its ends, and this is neither readily nor economically constructed, and, as a rule, is impracticable unless the stream is a very small one. If the river is already crossed by a dam, it may afford a satisfactory opportunity for measurement, provided it is free from leaks and its crest is horizontal. Old mill-dams are generally not water-tight and are more or less out of alignment, and therefore not very reliable for this purpose.

The technic of weir measurements and reductions is described in

detail in Part II., Chapter 6; for practical purposes of first investigations, the following method will yield sufficient results. Ascertain the length of dam crest by stadia measurement or triangulation, and the height of overflow by differential levels between water surface some thirty feet upstream of the dam and of dam crest, and take corresponding flow from Diagram 2, which gives the volume passing per linear foot of overfall over a wide flat-crested dam in cubic second feet for depths of overflow in tenths of feet.

Example.—Overflow of 1.2 feet over dam crest 236 feet long represents a flow of $4.35 \times 236 = 1026.6$ c. s. f.

Mill-dams, unless in very bad condition, may furnish valuable data as to flow; millwrights know how much power is required to operate their plant, the effective head can readily be measured, and, crediting the water-wheels with an efficiency of about 65 per cent. when of old pattern, the volume passing through them can be computed. Nothing is more strongly impressed upon the miller than shortage of water and the length of time during which he has to shut down to raise the pond, and the depth below the dam crest to which the pond lowers during low seasons after running the mill a certain period; each one of these facts may be utilized in determining the low flow of the stream and its duration.

The author determined the low flow on the Cannon River, Minn., in 1904 from such data collected at a mill which had operated some forty years with a water-wheel equipment representing the earliest American turbine styles, and the result was only about two per cent. lower than was afterwards found from measurements and gaugings extending over an entire year, and this discrepancy was chargeable to leakage through mill tail-race.

Where weir measurement is impracticable, a well-conditioned cross section of the stream is selected, its area found by soundings, and the velocity determined with which water passes through it by meters or floats. Part II., Chapter 6, treats the technical phases of these measurements and of the instruments, methods, reductions, and computations; the practical application for preliminary investigations is as follows.

Select a section about one hundred feet long on a straight stretch, with shores parallel and of gentle and even slope, containing no islands, visible rocks, or other obstructions, and where the water appears to pass at a nearly uniform speed throughout the entire width. Stretch two

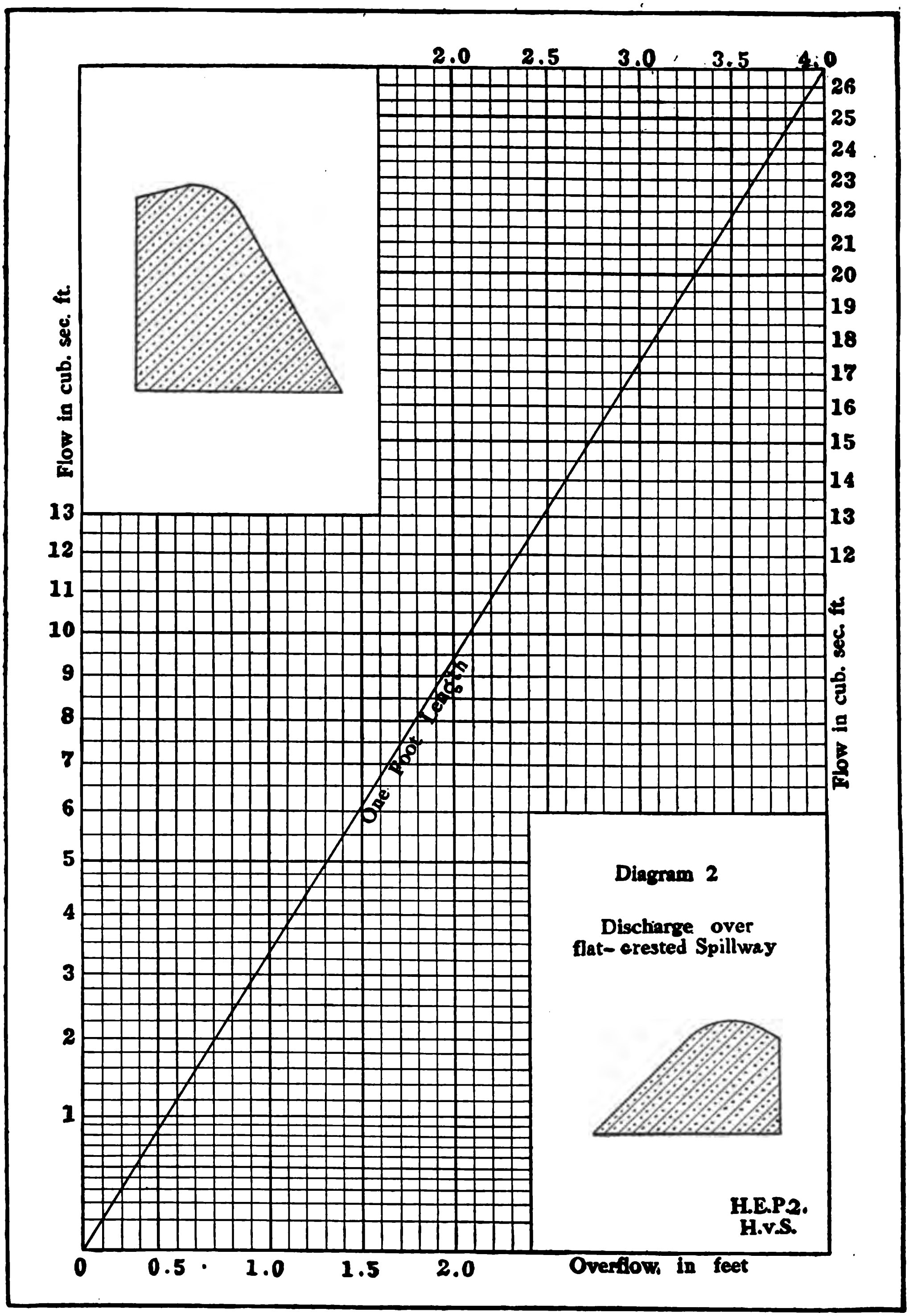


Diagram 2
Discharge over flat-crested Spillway
One Foot Length
Flow in cub. sec. ft.
Overflow in feet
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
1
2
3
4
5
6
7
8
9
10
11
12
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H.E.P.2.
H.v.S.

lines ($\frac{1}{4}$-inch rope) across the river, secured at shore and one hundred feet apart. Stretch a third line midway between these two and mark it off in ten-feet sections by tying on it alternately red and white bits of cotton ribbon. When the river is wider than one hundred feet, attach two guy lines to this centre line so that a boat may pass along without sagging it greatly out of straight line. Find the depth of water at each red and white marker along the centre line by differential level between water surface and river bed, by means of a levelling instrument reading on a rod held by a man, in boat, on river bottom. Collect chips of wood or pieces of lath or bark which can be recognized while floating, and have man in boat throw one at a time in the river some fifty feet above the upstream line, time the passage of float under upper and lower lines, also spot the red or white marker under which it passes; use as many floats as there are markers in the line, endeavoring to have at least one pass under each. Find the mean of the velocities observed, the product of 0.85 of this mean velocity and the cross section area represents the approximate discharge at that period for the purpose of preliminary investigation. Part II., Chapter 6, will deal with and describe measurements by meter, surface and rod floats, and of reductions, coefficients, etc.

ARTICLE 17.—*The fall* available for development must be found from the total fall existing in entire reach of stream to be controlled, that is, from the upper to the lower point to be affected by the development, less the fall represented by the slope in the upper pool, which may be from 0.05 to 0.5 feet per mile. The condition of river stage on which these fall determinations are based must be that which represents the flow to be utilized. If the consideration of backswell is neglected, the upper pool will extend beyond the expected limit, lands will be flooded which perhaps have not been secured, and, if there is an upper development or power site, the upper pool water will trespass on its tail waters.

When available fall is thus fixed, it remains to be determined whether all or only part of it is to be utilized, which will depend upon the topographical conditions as influencing location and character of development. The constancy of the fall as depending upon flow fluctuations must be carefully studied; on many streams the fall may all disappear during extreme floods.

ARTICLE 18. *Power Output.*—The unit of output of a hydro-electric development is the *electrical horse-power*, being representative of the power available for actual work.

The original energy is hydraulic power, which is converted by means of turbines into mechanical power, and this in turn into electric power; both of these transitions entail some loss of originally available energy, due to friction in one form or another. The amount of this loss from hydraulic to electric energy depends upon the type of the machines and their mode of operation, and in practice is generally considerably in excess of that which is claimed for them or even shown by tests. After the plant is in operation for a period, it is found that efficiencies of 76 per cent. for turbines and of 94 per cent. for generators may be obtained with proper equipment correctly installed. Based upon these the electric power realized is 72 per cent. of the hydraulic energy, or about 12½ cubic second feet with one foot fall represent one electric horse-power.

By the aid of Diagram 3 flow and fall may be readily converted into electric horse-power, or the flow required for certain output with fixed fall determined.

Example.—Flow 250 c. sec. ft., fall 22 ft. Output = 440 E.H.P. Fall 30 ft., desired output 100 H.P., required flow = 417 c. sec. ft.

Having found, by one method or the other, the monthly mean flow during a dry year, the volume on which maximum development may be based, which will be called the *power-flow*, is to be fixed upon.

The development plant, with exception of equipment, will generally cost nearly as much for a small as for a large output development, and, if for no other reasons, this is sufficiently important to seek the development into useful energy of the greatest possible portion of the entire flow during the dry year: the most complete utilization of this is the best development. However, the low month flow presents undisputably the maximum continuous volume which is available for twelve months; if any higher is taken, the deficiency must be made up, and the substance of the inquiry lies in the question, "How much can be added to the low flow, and from what source?"

The source is threefold: the market conditions may, and most generally do, call for current service only during a portion of the 24 hours; in that case the plant closes at the expiration of the operating period, and the natural flow may be accumulated by *pondage* above the dam during the remaining portion of the 24 hours, the non-operating period. This is accomplished by temporarily raising the height of the pond a foot or more through the fixing of flashboards along the crest; the ponded flow then becomes available, together with the natural flow,

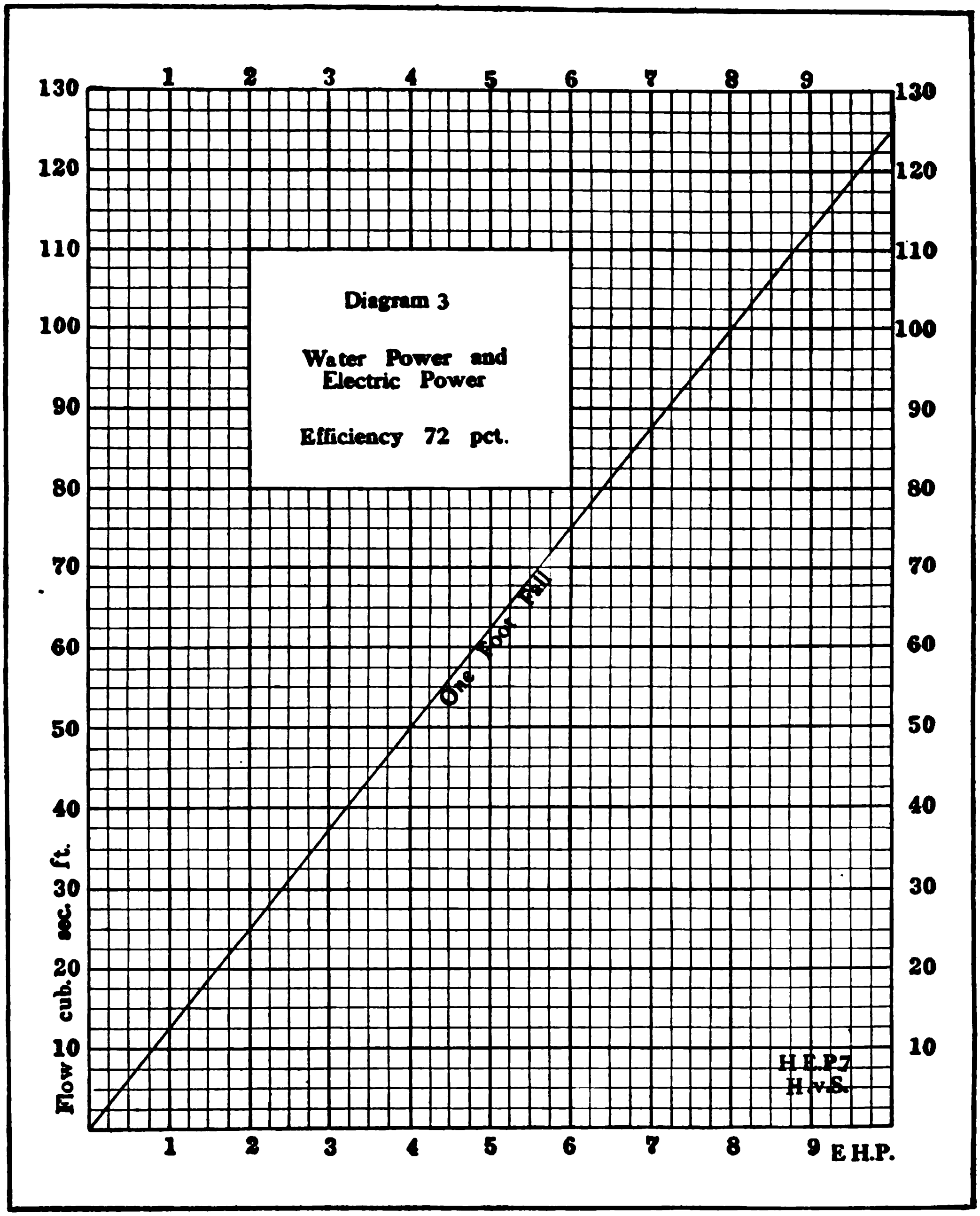


Diagram 3
Water Power and Electric Power
Efficiency 72 pct.
One Foot Fall
Flow cub. sec. ft.
E H.P.
H.E.P.7
H.v.S.

during the operating period.

For instance, the low monthly flow is....................250 c. sec. ft.,
the fall is.. 30 ft.,
the service is for interurban railway, or.................. 18 hours,
and the non-operating period therefore.................. 6 hours.
The ponded flow would be............ $250\times(6\times3600)=5{,}400{,}000$ c. ft.
and deducting 10 per cent. for leakage, the volume added
during the operating period would be $4{,}860{,}000\div(18\times3600)=78$ c.s.ft.,
representing..........$(78\div12.5)\times28.75$ (mean head) = 179.4 E. H. P.

In this connection it must be noted that a pondage reserve becomes available only by sacrificing some of the fall during part of the operating period; that is, the effective fall gradually drops as the pond is being drawn down. If the pond for above example were two miles long with a mean width of 200 ft., its area would be $10{,}560\times200=2{,}112{,}000$ sq. ft., and to accumulate the above 5,400,000 cu. ft. the surface will be raised $5{,}400{,}000\div2{,}112{,}000=2.5$ feet. The effective head will be 30 feet at the beginning and 27.5 feet at the end of the 18 hour operating period.

That a water power is always increased through the existence of a large upper pool or pond is a very common and erroneous belief, which comes down from the former-day mill practice when it was of no moment that the wheels stopped running for an hour or so, nor did a considerable difference in their speed cause any inconvenience or loss in power work; in fact the slow-motion-stone runs would be credited with producing the finest flour. The pond was indeed a great and valuable asset to the mill of old, but its present-day significance is radically different, and for the most desirable condition of a hydro-electric plant, its continuous operation, the pond above the dam has no power-enhancing value whatever, and may, then, lay claim to being desirable only for the purpose of safely collecting logs and ice before passing these through the proper sluices.

Pondage power increase involves, as stated already, decrease of head during the operating period, and the power output therefore fluctuates.

In the case of the example given the theoretical output at the starting of the plant is $(328\div12.5)\times30=787$ E.H.P., at closing plant is $(328\div12.5)\times27.5=722$ E.H.P., and from this it is evident that only the latter smaller power can be contracted for and that therefore the resourceful increase due to pondage in this case is not 179.4, as first found, but

in reality only the difference between $(328 \div 12.5) \times 27.5$ and $(250 \div 12.5) \times 30$ or $722 - 600 = 122$ E.H.P. In other words, the increase of $787 - 722$, or 65 E.H.P., cannot be contracted for, and, unless it can be otherwise used, or the contract has a blanket power-delivery clause by which the customer pays for whatever current is served to him, it is simply wasted.

A further reason why pondage power increase is often of less value than it is popularly credited with grows out of the fact that the highest efficiency of the power-generating equipment cannot be realized under conditions which differ greatly from those which are normal for their best performance. The power output of a turbine is as the square root of the cube of the fall and its speed as the square root of the fall, provided the flow equals the turbine's discharge capacity.

In the case of a development with the above-described pondage conditions, but a maximum head of 16 feet and minimum head of 13.5 feet, the output with the lower head is 77 per cent. of that with the higher and the speed with the lower head is 92 per cent. of that with a higher.

The suitable single turbine unit for 328 c. sec. ft. and 16 ft. fall is a 56 inch (see Diagram 38), its normal speed is 120 revolutions per minute, but when the head is lowered to 13.5 feet the speed is reduced to 106 r.p.m. If the generator is driven by 1 : 2 gearing the normal generator speed would be 240 r.p.m. and the reduced 212 r.p.m. This generator is designed to yield the desired output and the chief basis for such design is its speed; it is supplied with a certain number of poles in order to generate current of a fixed frequency of alternations, because any part of the armature passes a given number of poles in a minute's time with the assumed speed of revolution. The field coils and the armature windings are likewise determined from the assumed speed in order to generate the desired electro-motive force, and thus all the different characteristics are made to comply largely to the requirements of speed, as will appear more in detail in Chapter XI. Therefore a generator designed for a speed of 240 r.p.m. becomes not only inefficient when slowing down to 212 r.p.m., but the dropping of the voltage and of current output entail other and costly losses when the product is transmitted to a distant market.

From all this it appears that pondage as a supplementary flow source is of doubtful value when the available fall is 20 feet or less, and that the minimum head secured by pond-level fluctuations represents the maximum continuous operating head for which the generating equipment must be designed.

The most satisfactory source from which the low flood may be supplemented is storage, which refers to the accumulation in reservoirs of some of the excess over the normal flow volume during any or all of the periods when it occurs and the utilization of this stored supply during the low flow periods to maintain a flow higher than the natural low flow. The run-off may range from two tenths to two cubic feet per second per square mile of drainage area, without storage supplement and the lower run-off ratio must be taken as the available potential flow, but reservoir opportunities may exist where a considerable portion of the run-off which is in excess of this low volume may be stored to be drawn upon at will, whereby the low run-off and the potential flow may be increased many fold.

Storage is one of the most important features to be considered in connection with a hydro-electric development; during the past it has received practically no attention and many water power plants now in operation could be greatly enhanced in their earning capacities if storage were thoroughly investigated and provided where obtainable. It is one of the subjects of the conservation of natural resources, and an important one; its scope is the diminution of floods, the decrease of destruction to life and property caused by them, the improvement of river navigability and a utilization of a part of these large and now wasting and destruction-carrying flood waters in and for power development. There really is no larger question to deal with in water-power utilization and none which will return greater rewards.

This has been recognized by the older European nations some time ago. There water power has been given the name of "white coal" which emphasizes the modern appreciation of this natural source for all of mankind's needs as obtainable from energy. European water powers are now being developed, not hap-hazardly for certain interests, but for the most good to the communities which should, by reason of their geographical proximity to the power source, be first entitled to secure its benefits. Nor are they being developed on programs of smallest first cost and eventual large revenues without regard to the greatest practicable utilization of the opportunity, but on the contrary each power is studied, not for or by itself, but in relation to and in connection with the other power possibilities on the stream in question and the most resourceful conservation of the greatest portion of its flood flow.

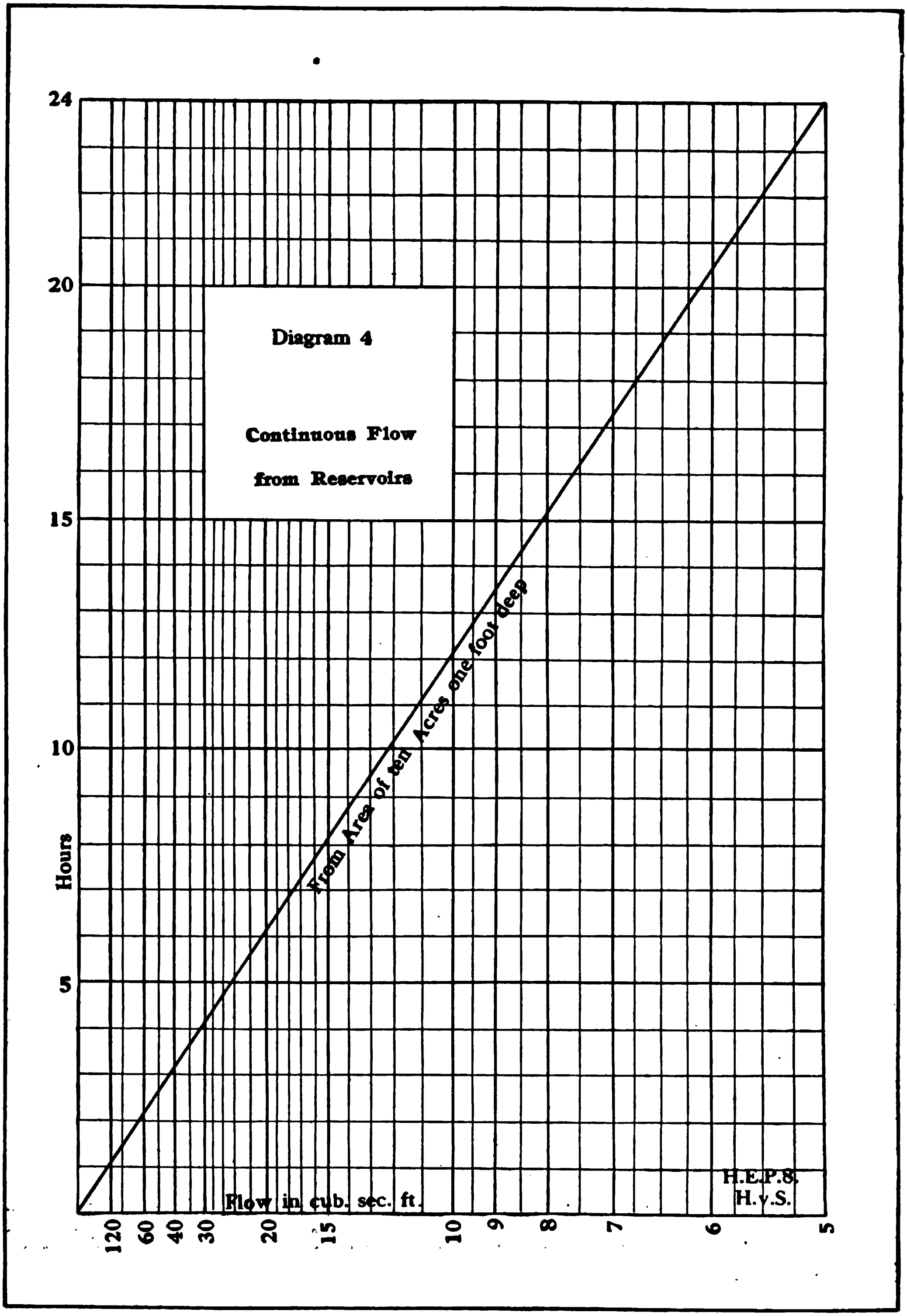


Diagram 4
Continuous Flow
from Reservoirs
From Area of ten Acres one foot deep
Hours
24
20
15
10
5
Flow in cub. sec. ft.
120
60
40
30
20
15
10
9
8
7
6
5
H.E.P.S.
H.v.S.

In discussing storage the acre is the most convenient area unit and the acre-foot the most suitable storage water measure. One acre contains 43,560 square feet, and, as one day consists of 86,400 seconds, one acre of land covered by one foot of water, an acre-foot, practically represents a 24 hour flow of half a cubic second foot.

Diagram 4 gives the equivalents in continuous flow from various storage volumes; as, for instance, the flow for ten hours from ten acre-feet equals 12 c. sec. ft. The practical application of flow supplement from storage is considered in Article 51, treating of the scope of the development and giving a complete analysis of the method by which the most resourceful development from the natural flow, storage, and auxiliary power supplements may be discovered.

CHAPTER III

FEASIBILITY AND PRACTICABILITY

The feasibility and practicability of a project are not entirely established because the power and market are available: questions of Government control on the stream, of State laws regulating the use of water, of riparian rights and land titles must be investigated and settled, while the practicability of economical development should be established.

ARTICLE 19.—*The United States Government exercises constitutional control* over all navigable waters, and the question as to the navigability of a watercourse rests with the Congress. When provisions are made by Congressional legislation to examine a river for the purpose of determining whether the commercial interests and the physical conditions warrant navigation improvements, the stream passes under the control of the War Department, and no works of any description for the development of power along that river, or so much of it as is covered by the act, can be erected without the consent of the Secretary of War, nor can such works be erected on rivers where navigation improvements have been made by the Government without a like consent. The policy of the War Department is to grant such permits where no interference with navigation works is threatened, reserving, however, a revocable authority and claiming approval of plans and of construction.

ARTICLE 20.—On navigable rivers *the title to shore lands carries* to low-water line, while on non-navigable streams it goes to the middle of the watercourse, securing to the owner the use of the water within the boundaries of his lands for floatage, power purposes, and fisheries, but no portion of the volume can be diverted from its normal course or put to such a use that the natural flow or fall pertaining to any land not owned by him is thereby periodically or permanently diminished or changed. The *State retains control* of the land under the water, and several have enacted laws reserving the approval of any permanent structures such as are required for power development; this right, however, does not include the power of prohibition of such works, but merely of securing safety of structures and of provisions guaranteeing non-interference with

the floatage of timber and the passage of fish to their accustomed breeding-grounds. Some of the States have constructed navigation canals along certain rivers, and in this connection have reserved control of a portion of the flow and of certain reservoirs maintained for the feeding of the canal system; water-power development on such streams must conform to statutes enacted for the protection and maintenance of such State canals. The Western States have legislated on the use of water of streams for the purpose of guaranteeing its equitable distribution to adjacent land for irrigation, and in these the permit of the respective authorities must be secured for the use of any volume for power purposes; and, finally, the Federal Government has taken possession of certain streams in the arid regions for purposes of national irrigation projects, and from these all other utilization of their waters is excluded.

The legal requirements and limitations relating to water-power development expressed in Federal and State statutes in the United States, Canada, and Mexico are now being compiled by the author, and will be published in the near future.

ARTICLE 21.—*The practicability of an economical development* is solved only after the programme has been fixed upon, the plant designed, and estimates made, all of which is treated in detail in Part II.; in some cases, however, difficulties to be met may be so apparent and the great expense of overcoming them so patent that the undertaking may be pronounced prohibitive without the necessity of detail engineering studies. Market and power capacity enter into this problem, and in fact their proper consideration will frequently terminate the entire inquiry. Many excellent water-power opportunities are undeveloped to-day and will remain so for some time to come, because there is no market available for their output, nor is there any in sight for many years to come unless electric power is utilized by railroads generally for their passenger business, which is, no doubt, among the probabilities of the near future. In some cases even the best available development site reveals, upon preliminary examinations, such difficulties, as to length of required dam and unstable material at its logical location, that it becomes at once evident the cost of the necessary works will be out of all proportion to the power which may be secured; and, finally, the constancy of the output may be proved to be entirely void of any reasonable guarantee of that continuity which must underlie the enterprise which proposes to meet binding contract obligations.

ARTICLE 22. — *The investment balance*, or the summation of the capital outlay and the returns promised by the enterprise, is the quintessence of the analysis of a hydro-electric project; if this, being based upon authoritative facts, makes a satisfactory showing, and the development is found to be feasible, its realization is a certainty.

The treatment of this subject is the same as that of any other commercial proposition: charges and revenue are the components. The first consist of interest on investment, sinking fund, maintenance, operation, depreciation, taxes, and insurance; the second, of receipts from sale of product. An annual charge of 8 per cent. on capital investment will meet the interest of 5 per cent. and the retiring of a bond issue representing the investment in 20 years. Maintenance and depreciation should be charged at 2 per cent. of the cost of the works and 3 per cent. of that of equipment; operation cost of generating, transmission, and distributing plant may be estimated on a personnel of two shifts, each composed of an operator and assistant and of one lineman for day service, which is required for a plant whether of 500 or 5000 horse-power output, or a line 5 or 25 miles long; the wages allowed should not be less than $90.00 for the operators, $60.00 for assistants and $75.00 for linemen. The maintenance charge will cover oil, waste, wire, insulators, and all other needed repair material. Taxes are generally 2 mills on a $\frac{3}{4}$ valuation. Insurance is not always placed on hydro-electric plants, as the fire risk is small.

Tabulating these charges for a plant of 1000 horse-power with ten-mile transmission line, the fixed items are the cost of the generating equipment, which will be about $20.00 per horse-power, cost of transmission equipment (transformers) $6.00 per horse-power, and of substation at market end $2000.00; no estimate is made for distribution and service lines and equipment, and the receipts from current are assumed to be its wholesale value at the substation, or $30.00 per horse-power-year.

The statement for one horse-power ratio will be

Charges:	
Interest on sinking fund	$ 4.28
Maintenance and depreciation	1.59
Operation	8.22
Taxes and insurance	0.80
Income	30.00
Balance	15.11

This balance must meet interest and sinking fund at 8 per cent. and maintenance and depreciation at 2 per cent. of the cost of the works, consisting of a dam, intake, power-house, and tail-race, of lands, right of way, taxes, and discount from face value of securities and service charges, and represent a principal of about \$140.00 per horse-power; or, in other words, the cost of these, the works, lands, etc., must not exceed \$140.00 per horse-power in order that the enterprise may meet fixed charges, while a smaller cost will leave a corresponding surplus. If, for example, the cost of items not above included aggregates \$70,000, of which \$50,000 is cost of works, then the statement is:

Cost of complete development	\$123,500.00
Capital investment	137,200.00
Interest and sinking fund at 8 per cent	11,000.00
Maintenance and depreciation	2,590.00
Operation	8,220.00
Taxes and insurance	1,800.00
Income	30,000.00
Surplus	6,390.00

or about 5 per cent. on the investment.

The operating charge is the largest item, and, since this does not materially increase with a greater output up to about 5000 horse-power, the larger capacity developments will show a greater surplus.

The income from current is here taken rather low, being for ten-hour motor service at a rate of 1.3 cents per kilowatt-hour, and for eighteen-hour traction service only about 0.6 of a cent, both of which are considerably below average values; however, the same line of investigation adapts itself to the probable value of the output as found from market investigations, and, when investment required is known, the balance statement can be safely made up along these lines.

Diagrams 5, 6, and 7 show the fixed charges per horse-power for plants of varying capacity and total investment, and by the aid of Diagram 8 the horse-power per year values can readily be converted into kilowatt-hour rates for the three classes of current service, light, motor, and traction.

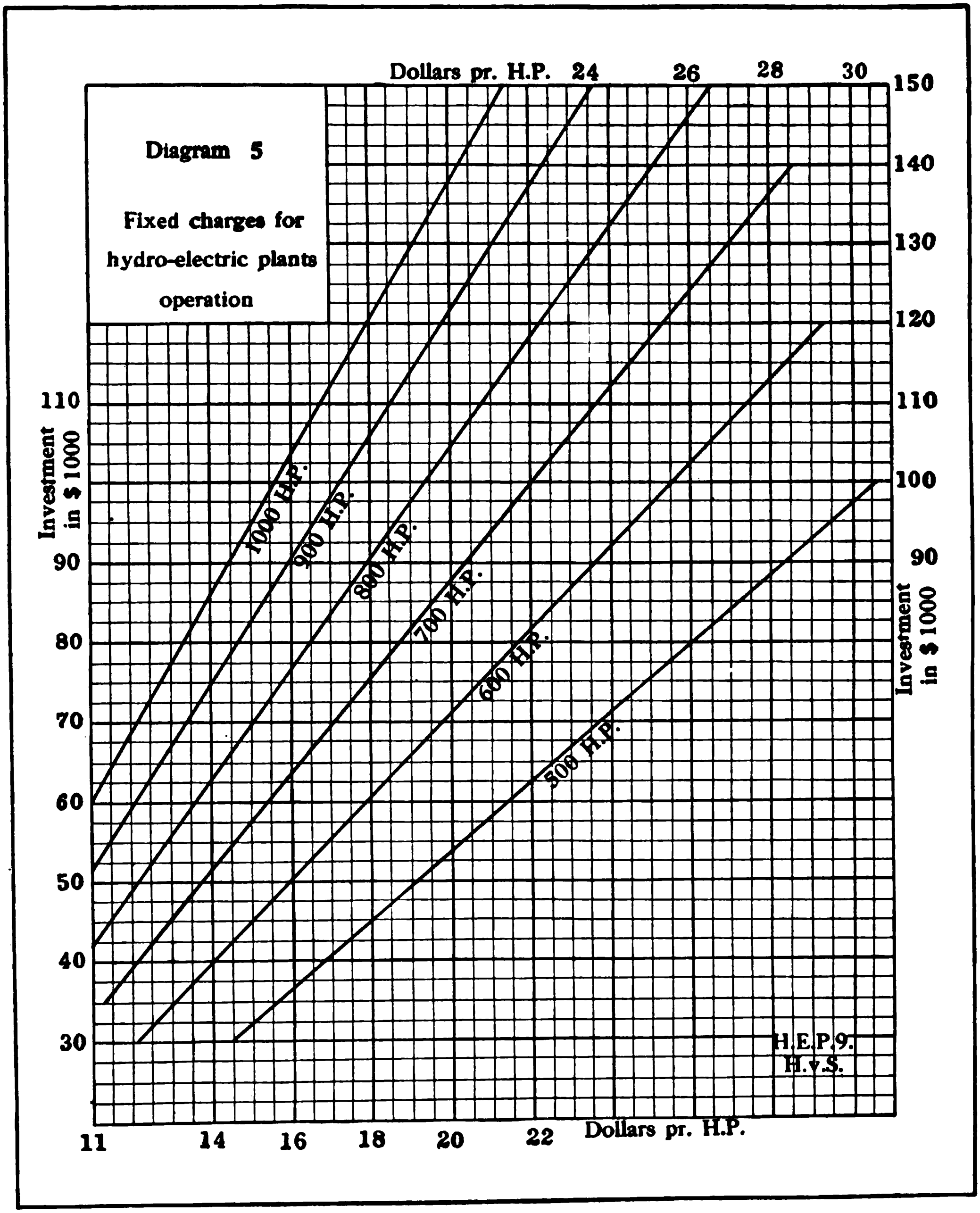


Diagram 5
Fixed charges for hydro-electric plants operation
Dollars pr. H.P. 24 26 28 30
150
140
130
120
110
100
90
80
70
60
50
40
30
Investment in $ 1000
1000 H.P.
900 H.P.
800 H.P.
700 H.P.
600 H.P.
500 H.P.
H.E.P.9.
H.v.S.
11 14 16 18 20 22 Dollars pr. H.P.

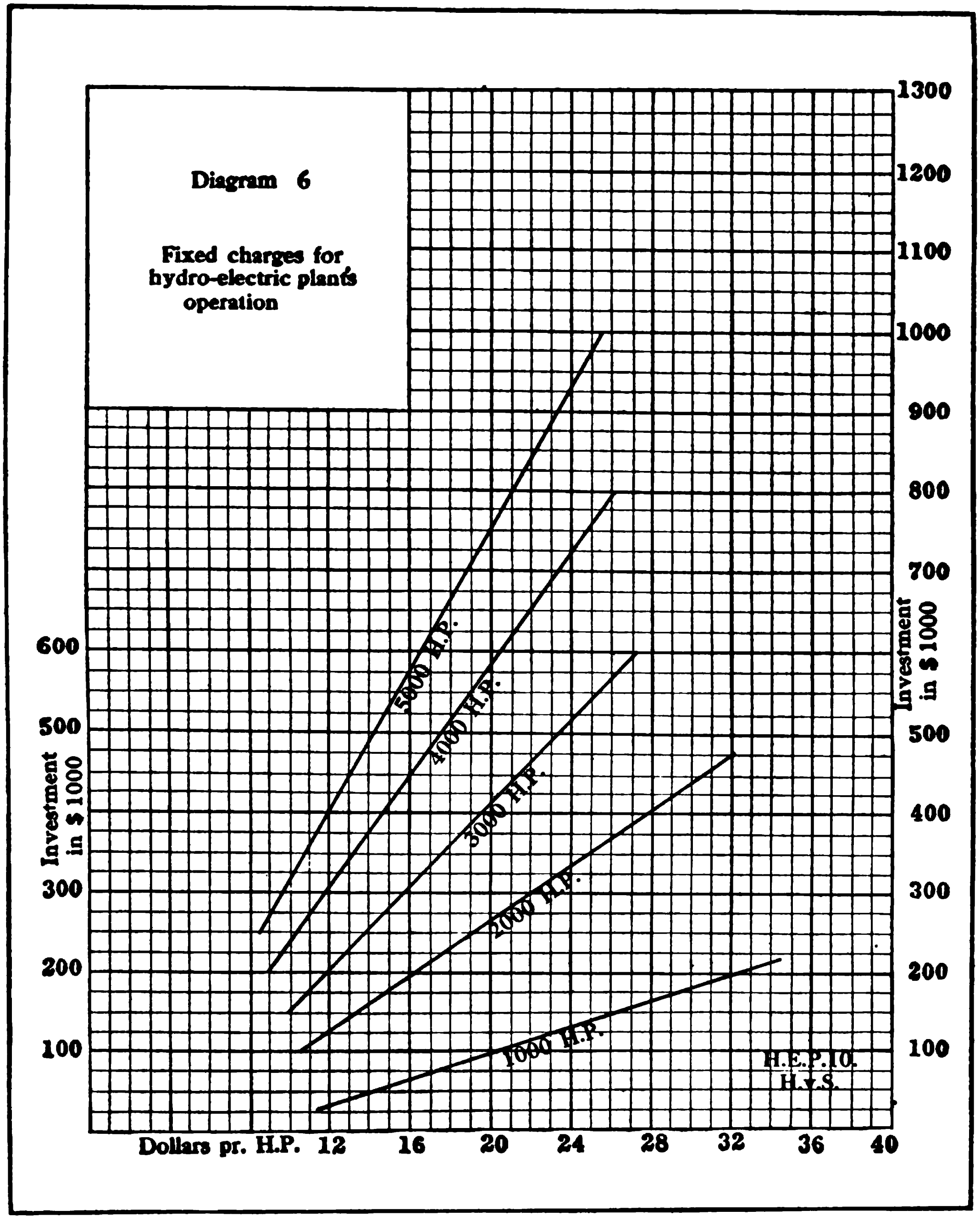


Diagram 6
Fixed charges for hydro-electric plants operation
5000 H.P.
4000 H.P.
3000 H.P.
2000 H.P.
1000 H.P.
H.E.P.10.
H.v.S.
Investment in $ 1000
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
Dollars pr. H.P. 12 16 20 24 28 32 36 40

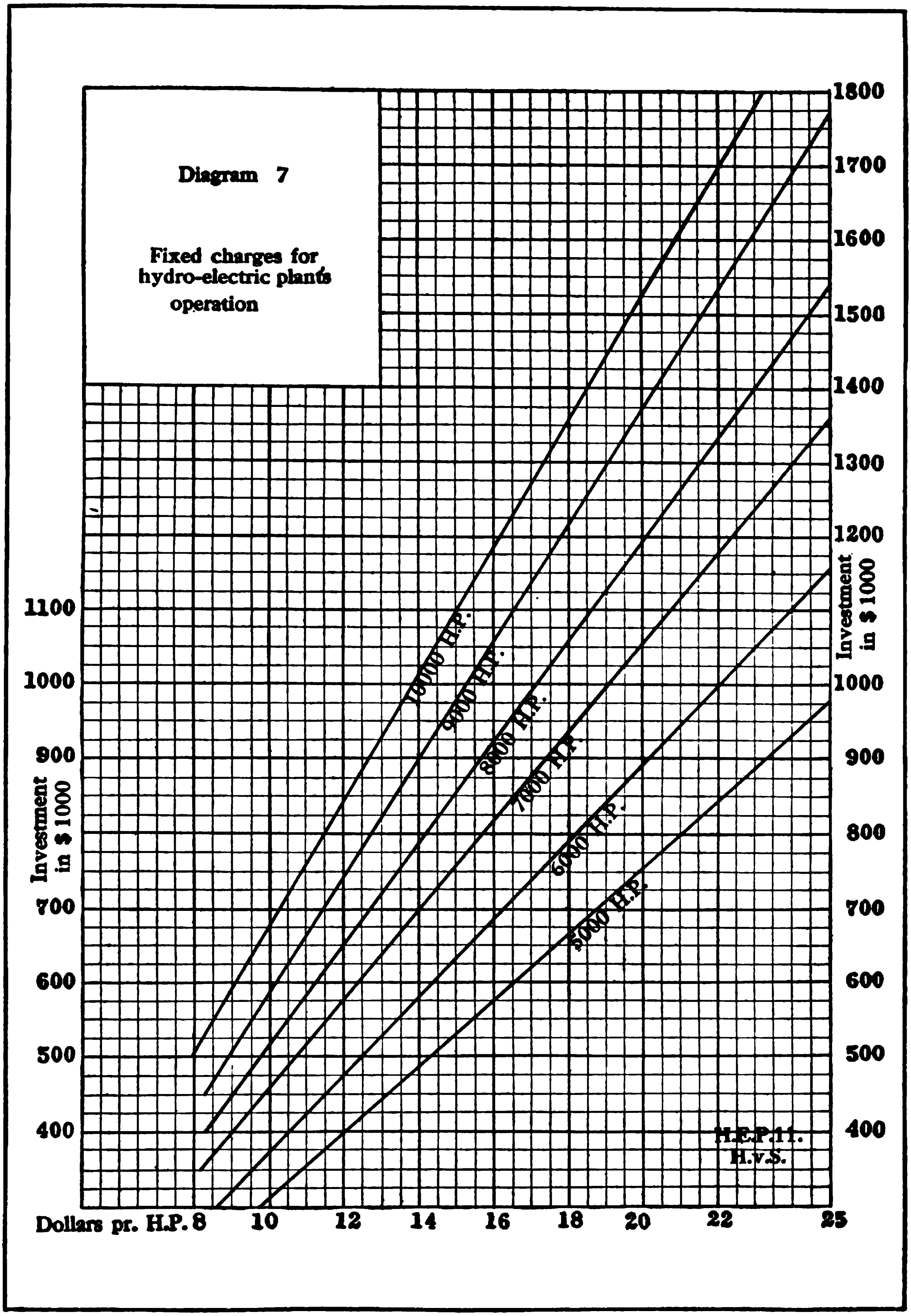


Diagram 7
Fixed charges for hydro-electric plants operation
10000 H.P.
9000 H.P.
8000 H.P.
7000 H.P.
6000 H.P.
5000 H.P.
Investment in $ 1000
1100
1000
900
700
600
500
400
1800
1700
1600
1500
1400
1300
1200
1000
900
800
700
600
500
400
Dollars pr. H.P. 8
10
12
14
16
18
20
22
25
H.E.P.11.
H.v.S.

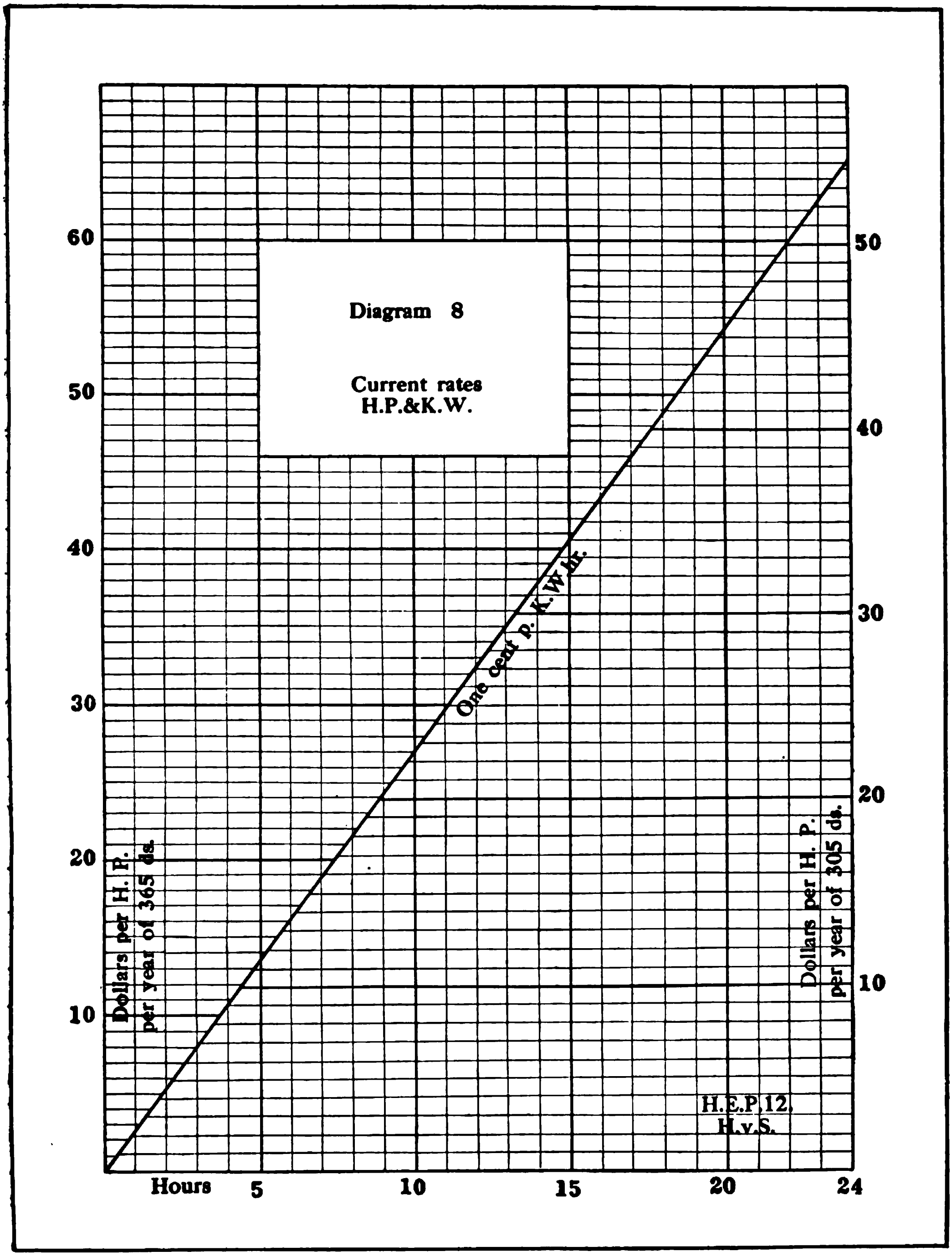


Diagram 8
Current rates
H.P.&K.W.
One cent p. K.W.hr.
Dollars per H. P.
per year of 365 ds.
Dollars per H. P.
per year of 305 ds.
60
50
40
30
20
10
50
40
30
20
10
Hours 5 10 15 20 24
H.E.P.12.
H.v.S.

CHAPTER IV

COST OF DEVELOPMENT

THE COST of the development can be found correctly only from estimates based upon a well-defined programme and the plans and detail designs of the required structures, all of which is treated exhaustively in Part II. Only some general rules and guides are given here, by which an approximate first estimate of such cost may be found.

ARTICLE 23.—The cost of the *dam*, only the spillway proper being here considered, depends not only upon the type to be built but also upon the conditions of flow and character of river bed. A reasonable allowance must be made for controlling the flow during construction period, which may require erection of coffer-dams of more or less substantial design and the operation of a pumping plant. Where the river is shallow and the possibilities of a material rise during construction period are remote, this item may not be very important; sheet piling often proves sufficient to exclude the flow from the area, the seepage being accumulated in one sump and thence removed by moderate pumping. Again well-constructed timber cribs loaded with rock may be required to guard against flooding and a large capacity pumping plant needed to perform continuous duty. Sheet piling, where gravel and boulders prevail, is best of interlocking steel material, which will cost from $1.50 to $3.00 per foot length of piles in place, depending upon the depth to which they are to be driven. In rock or hard bottoms timber cribs must be employed to coffer the desired area; these are constructed of square timber suitably framed, from 6 to 10 feet wide, filled with rock and puddling material; if water is shallow, rock diking with puddling placed on outside may answer the purpose. The characteristics of the flow, of material in river bed, and probable duration of construction under such safeguards must decide the means to be employed; at any rate, the item "control of flow" is an important one, and may be from a few hundreds to several thousands of dollars. For some dam constructions it will be less than for others, even on the same site; with a structure, for instance, consisting of piers rather than a continuous mass of masonry the completed portion can readily be utilized for water-way while the

remaining is being built; like methods can be employed with concrete-steel dams.

This brings us to the type of dam itself, which should be chosen because of its peculiar fitness to the conditions and purpose as well as for considerations of first cost. Part II. treats this point in all its practical phases.

Diagrams 9 and 10 give quantities required for masonry, concrete, and concrete-steel dams of different heights and for unit length of ten feet; the cost can be found for different values of labor and material from Diagram 11.

The dam should contain waste-gates or sluices and perhaps flash-boards, which must be covered in estimate.

This cost, as given on diagrams, does not cover the construction in river bed required to carry the dam, safeguard it against underwashing or the downstream area against scouring. In soft locations a pile foundation, cut-off walls or curtains, and apron construction will be required, the extent and cost of which depend solely upon the character of the material on which the dam is to rest. Again reference for details is invited to Part II.

Diagram 12 gives quantities for foundation, cut-off, and apron construction in alluvial locations for different heights of dams and for lengths of ten feet.

And, finally, the dam terminates in abutments, unless the site is a rock gorge into which the dam structure may be built; Diagram 12 also gives quantities required for two abutments of concrete-steel design for dams of different heights.

Compiling the cost of the dam in alluvial river bed, with small flow and no floods during construction period, height above river bed 30 feet, length between abutments 250 feet, labor being $0.20 per hour, material for cub. yd. of xxx concrete $4.75, forming timber $25.00 per 1000 ft. b. m., and re-enforcing steel 3c. per pound.

Item 1.	Controlling flow:		
	Coffer-dam or sheet piling		$2,500.00
	Pumping, 200 days at $5.00		1,000.00
Item 2.	Preparing bed, excavation, etc		500.00
	Foundation, cut-off and apron dam:		
	Rubble masonry	$36,300.00 or	
	Cyclopean concrete	28,117.00 or	
	Concrete-steel	22,100.00	
Item 3.	Abutments		2,060.00
Item 4.	Waste- or sluice-ways and gates depending upon flood flow volume.		

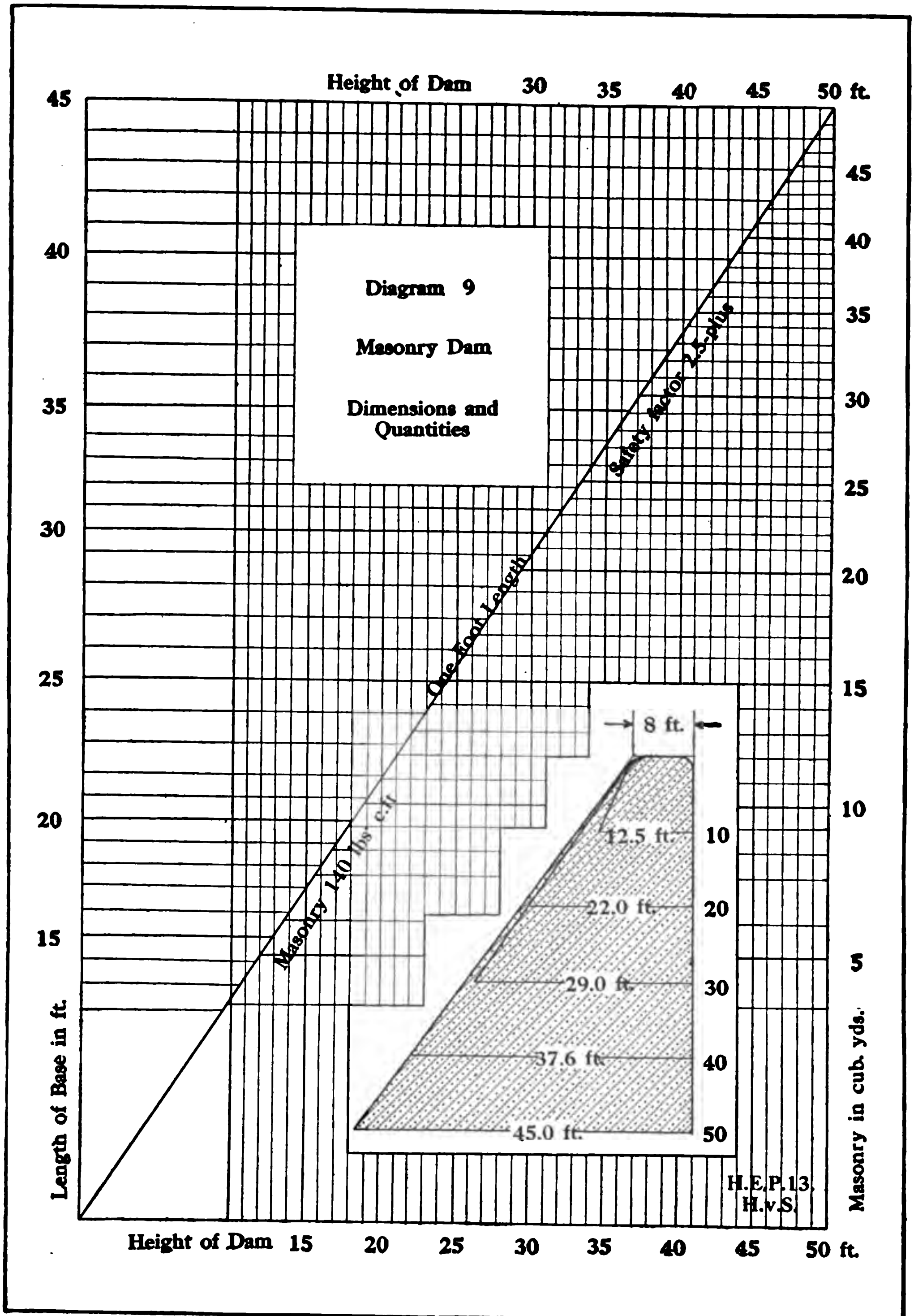
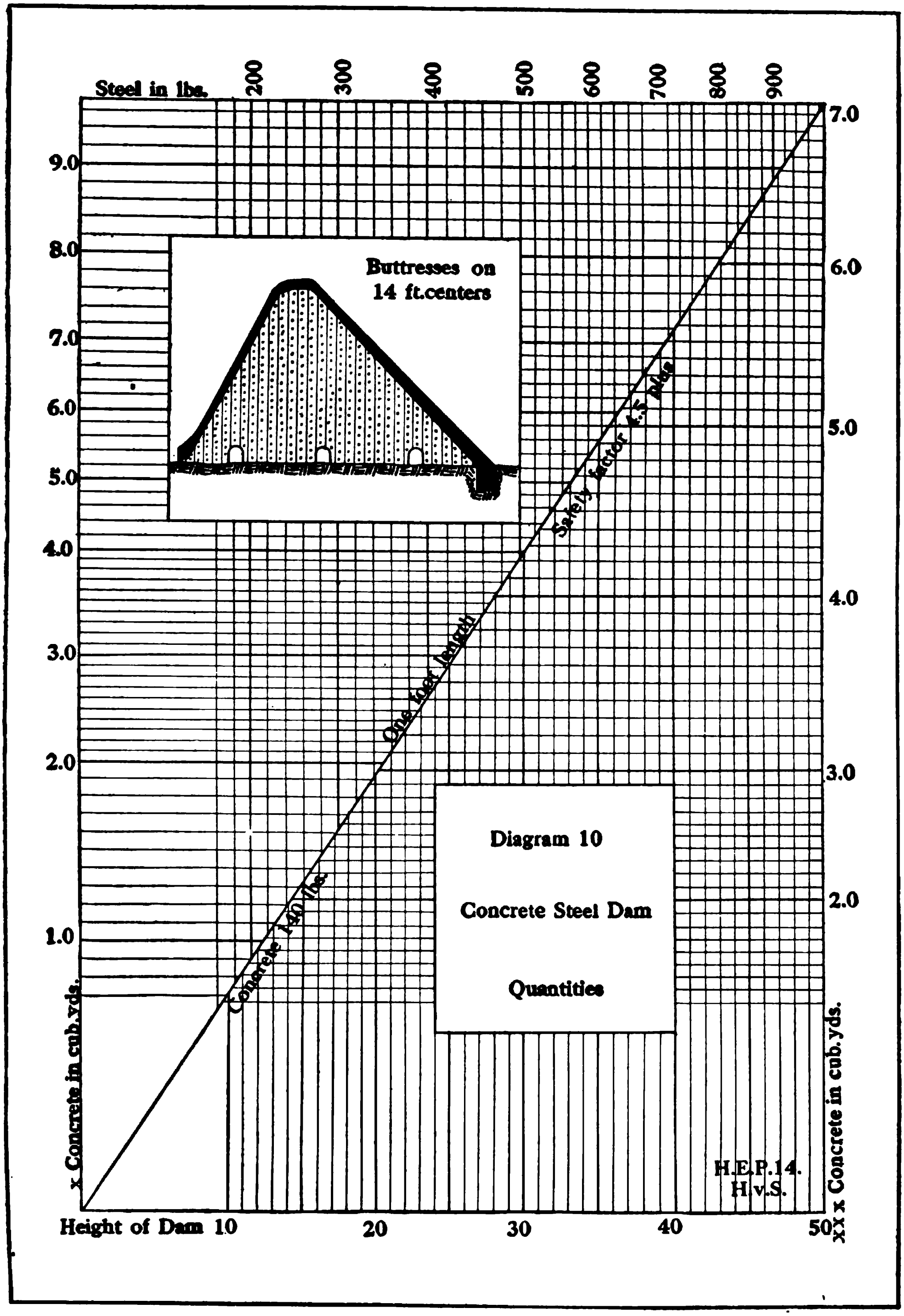
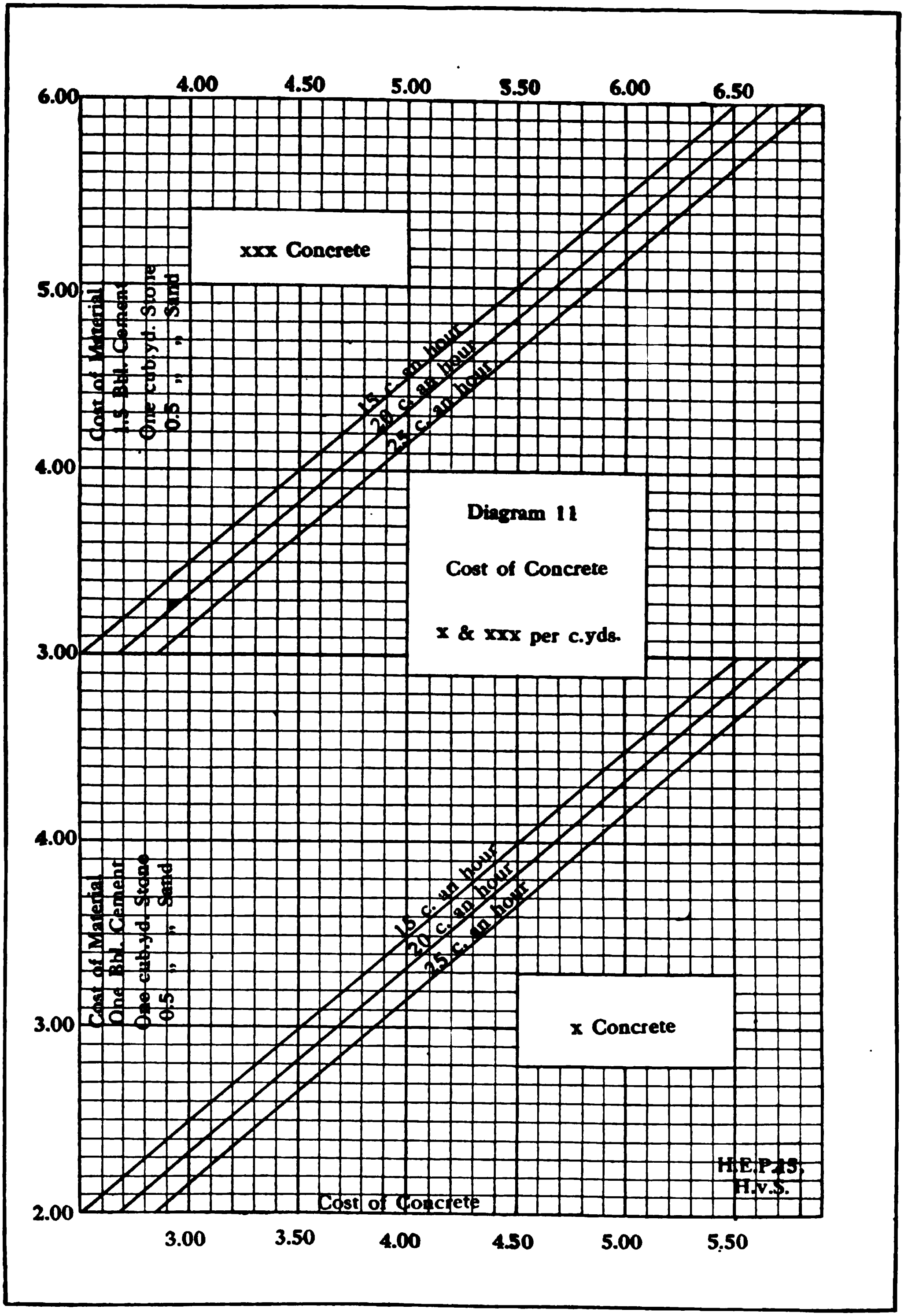
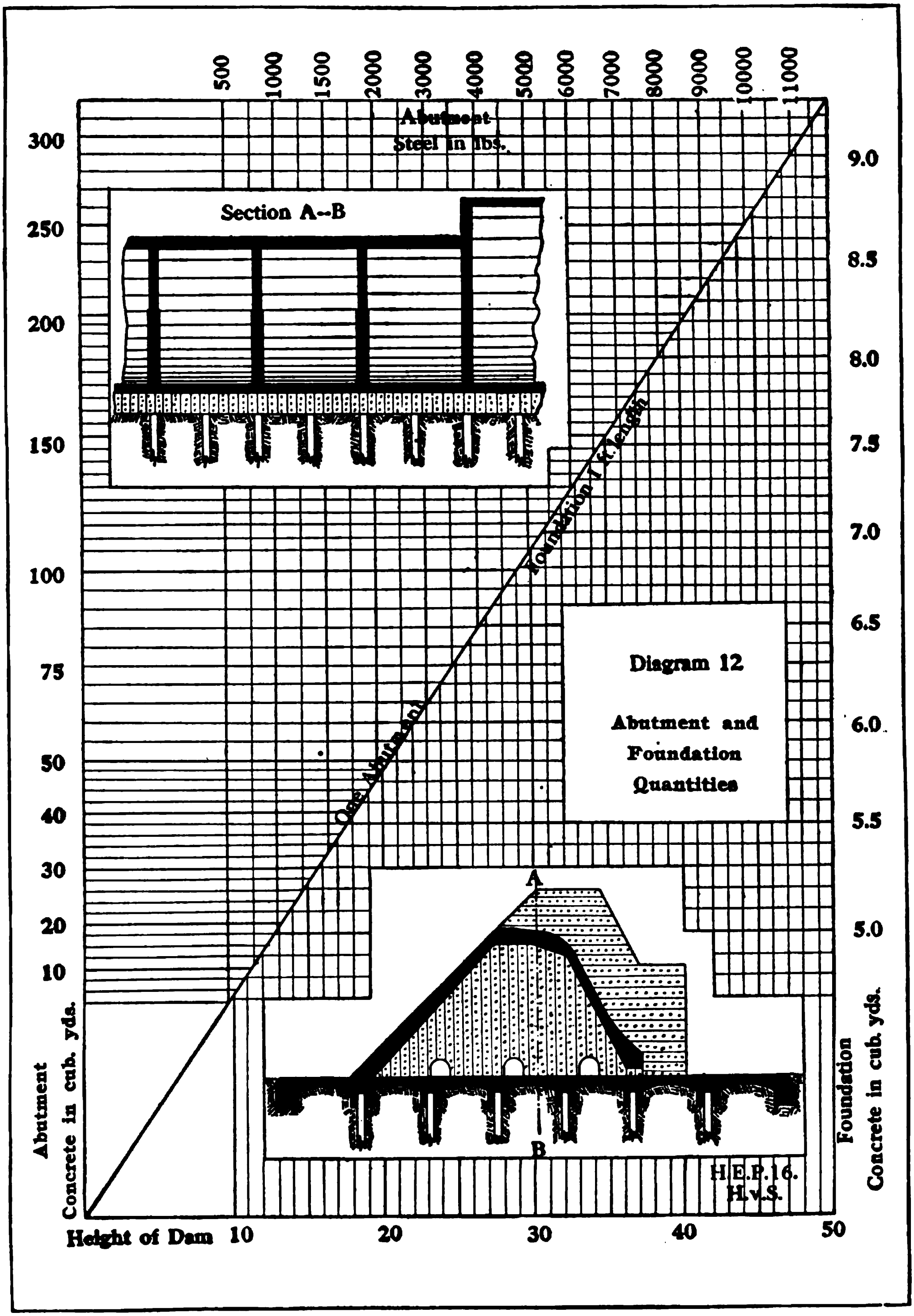


Diagram 9
Masonry Dam
Dimensions and Quantities
Height of Dam 30 35 40 45 50 ft.
Height of Dam 15 20 25 30 35 40 45 50 ft.
Length of Base in ft.
45
40
35
30
25
20
15
Masonry in cub. yds.
45
40
35
30
25
20
15
10
5
Safety factor 2.5 plus
One Foot Length
Masonry 140 lbs c.ft
8 ft.
12.5 ft.
22.0 ft.
29.0 ft.
37.6 ft.
45.0 ft.
10
20
30
40
50
H.E.P.13.
H.v.S.


Steel in lbs.
200
300
400
500
600
700
800
900
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
7.0
6.0
5.0
4.0
3.0
2.0
Buttresses on
14 ft.centers
Safety factor 4.5 plus
One foot length
Concrete 140 lbs.
Diagram 10
Concrete Steel Dam
Quantities
x Concrete in cub.yds.
xx Concrete in cub.yds.
H.E.P.14.
H.v.S.
Height of Dam 10
20
30
40
50


xxx Concrete
Diagram 11
Cost of Concrete
x & xxx per c.yds.
Cost of Material
1.5 Bbl. Cement
One cub.yd. Stone
0.5 " " Sand
15 c. an hour
20 c. an hour
25 c. an hour
Cost of Material
One Bbl. Cement
One cub.yd. Stone
0.5 " " Sand
x Concrete
Cost of Concrete
H.E.P.&S.
H.v.S.
6.00
5.00
4.00
3.00
4.00
3.00
2.00
4.00
4.50
5.00
5.50
6.00
6.50
3.00
3.50
4.00
4.50
5.00
5.50



Abutment
Steel in lbs.
500
1000
1500
2000
3000
4000
5000
6000
7000
8000
9000
10000
11000
Section A--B
One Abutment and Foundation 1 ft. length
Diagram 12
Abutment and Foundation Quantities
A
B
H.E.P.16.
H.v.S.
Abutment
Concrete in cub. yds.
300
250
200
150
100
75
50
40
30
20
10
Foundation
Concrete in cub. yds.
9.0
8.5
8.0
7.5
7.0
6.5
6.0
5.5
5.0
Height of Dam
10
20
30
40
50

Article 24.—*The diversion works,* by which the water is carried from dam to the power station, are next in line. These may be canal, flume, or pipe line, depending upon the volume and distance over which the water must be conducted. If no diversion is required, these are, of course, unnecessary. These structures involve excavation of rock or loose material, timber or concrete lining, slope paving, timber framing, wood stave, steel plate, or concrete-steel conduits, pipe-line anchorages, flume supports, intake works, head-gates, culverts, waste- or sluice-ways, and gates and forebays with bulkheads, all of these being treated specifically as to design and construction in Part II.

Excavation cost of rock depends upon its character, and may vary from 75 cents to $1.50 per cubic yard, also that of loose material from 25 to 50 cents per cub. yd.; timber lining of canals, with timber at $20.00 per 1000 ft. b. m., costs $4.50 per square yard, concrete lining $5.00 per sq. yd., slope paving $1.25 per sq. yd., timber framing of flumes costs $40.00 per 1000 ft. b. m. The cost of pipe line at present is

	36-inch.	48-inch.	60-inch.
Wood stave	$4.00	$6.00	$8.00
Steel plate	6.00	8.00	10.00
Concrete-steel	5.00	7.00	9.00

per linear foot of pipe, to which must be added cost of delivery, and putting in place, painting, etc.

Cost of head-gates, waste- and sluice-ways, etc., depends upon character of design and size of conduit.

The canal prism should be of an area sufficient to pass the flow at a velocity not exceeding three feet per second; the diameter of pipe conduits depends upon the ratio of head which can be economically expended in friction. The designs of all diversion works should be based upon appropriate hydraulic theorems.

Article 25.—The *power-house* is the last of the principal structures. Its location is determined by the development programme; it may be at the end of the dam or inside of the spillway, immediately below or at the terminal of diversion canal or pipe line. The design is fixed by the method of bringing the water to the turbines, the dimensions by number of power units it is to contain. Whether it is recommendable to let the water enter the power station freely or by means of feed pipes depends upon the height of fall, volume of flow, and topography at chosen power-house site. In both cases the structure consists generally of three parts,

—foundation, substructure or pit, and superstructure in which turbines and generators are housed. Foundation must adapt itself to the character of the material at the site, pit to volume of flow, height of fall and of backwater, and superstructure to power equipment. The walls and floors should be of masonry, monolithic concrete, or concrete-steel. Wherever permissible the water should be taken to turbines in conduits, as the power-house required for such an arrangement will be considerably less costly than where the water enters freely; and, for the same reason, if otherwise recommendable, the power units should be rather large than small, as the length of the power-house materially depends upon this condition. Especially is this true where water enters freely, since the structure in that event performs, in a sense, the functions of a dam, and the foundation and pit structures must in that case be designed not only for the duty of supporting vertical loads but also to resist horizontal pressures, which adds considerably to dimensions. The power-house should be readily approachable by the best available means of transporting the heavy equipment; ample room should be provided on the operating floor, so that each machine can be readily dismantled, repaired, or removed; a power traveller, by which parts of equipment can be handled, should be provided. There should be no stinting of light, and the roof had best be of the most substantial and fire-proof character.

Diagram 13 gives quantities for power-house per power unit length and for varying heights of fall, both for structures into which water enters freely and where it is conducted to turbines by feed pipes; the same diagram shows quantities required for foundations for structures in alluvial locations.

ARTICLE 26.—In addition to these works there may be required *reservoir embankments*, in the event that a part of the stream valley must be closed; in fact, this is the most frequent condition. These embankments are a continuation of the spillway, but rise sufficiently higher to guard them against possible overflow, the spillway proper being designed of sufficient length to pass the greatest probable flood volume at a safe height. These structures may be of earth or concrete-steel, depending largely upon the availability of suitable material for the former type. Reservoir banks or bulkheads partake of the importance of the spillway, and require the greatest care of design and still more so of construction. This consists in preparation of surface on which they are placed by a complete removal of all vegetable growth, roots, and stones, loosening

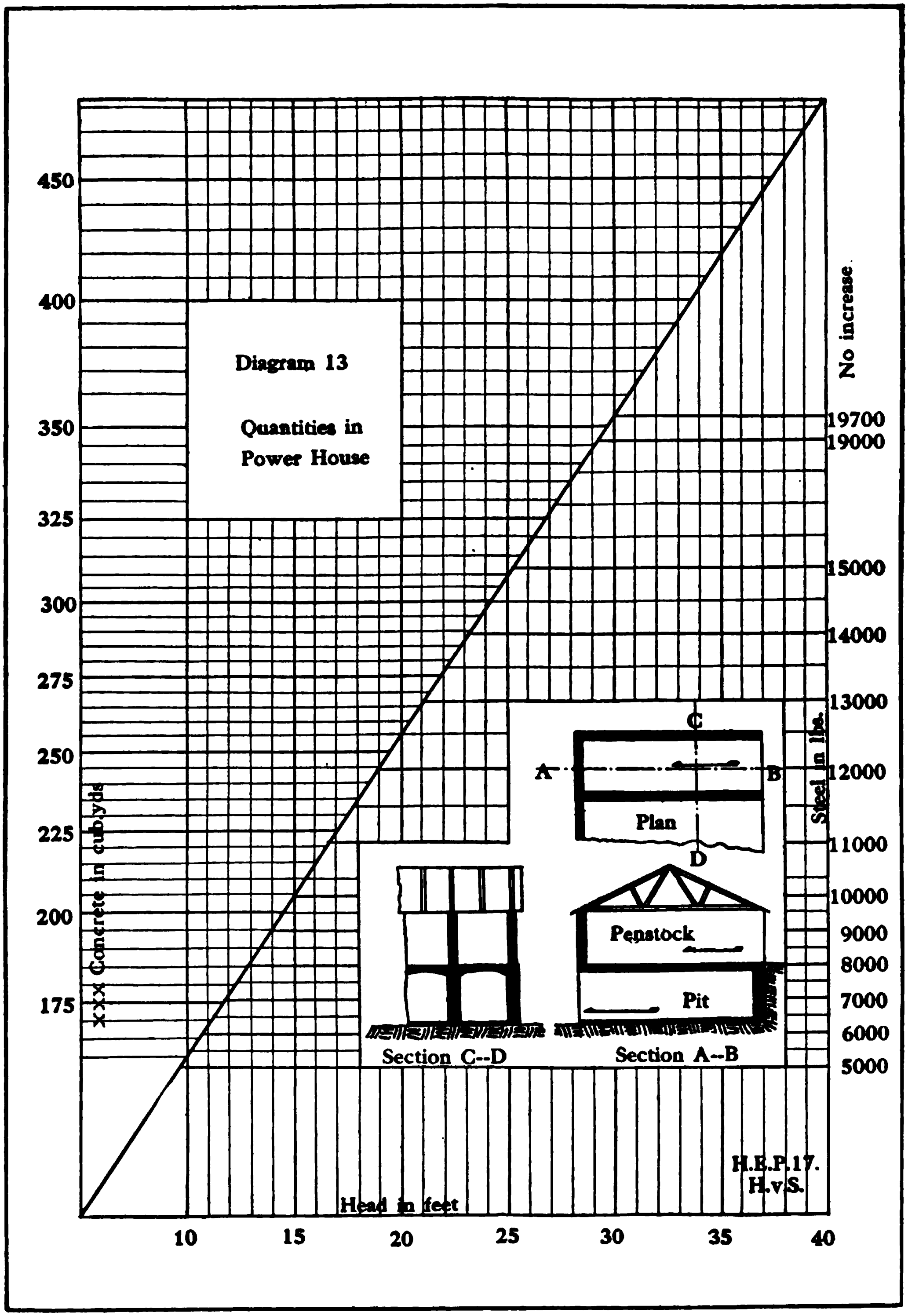


Diagram 13
Quantities in Power House
450
400
350
325
300
275
250
225
200
175
××× Concrete in cub.yds
No increase
19700
19000
15000
14000
13000
12000
11000
10000
9000
8000
7000
6000
5000
Steel in lbs.
C
A
B
Plan
D
Penstock
Pit
Section C--D
Section A--B
H.E.P.17.
H.v.S.
Head in feet
10
15
20
25
30
35
40

the soil to a considerable depth; constructing a core wall, which must penetrate into impermeable material, and compacting the material of which the bank is to be constructed in thin horizontal layers in a wet and plastic condition. Core walls should be of concrete and the banks of puddling material, being a proper mixture of clay, sand, and fine gravel which will pack solid when damp. This construction involves excavation at 25 to 50 cents, concrete at $5.00 to $6.00, and compacted earth fill at 35 to 50 cents per cub. yd.; the upstream upper slope should be paved at $1.25 per sq. yd.

Diagram 14 gives quantities for earth reservoir embankments with concrete core wall in lengths of one foot and for different heights.

Where the ground material is hard, reservoir bulkheads of concrete-steel construction may take the place of earth embankments when suitable material for these is not available. The quantities, in one-foot lengths, are given on Diagram 15.

This completes the works which are generally required.

Article 27.—The *power equipment* consists of water turbines, with governors and draft tubes, and of electric generators, exciters, and switchboards. When generators are coupled to turbine shafts and the units are of standard type, an estimate of $20.00 per horse-power will generally cover its cost; when turbines have to be geared to generator shafts, the cost per horse-power will be $24.00.

Article 28.—The last item comprises the *transmission line* and equipment. As a rule, a wooden pole line is recommendable, poles being 35 feet long and set 106 feet apart; a single circuit will be sufficient excepting for large outputs. Such a line requires cross-arms, pins, insulators, and three strands of bare copper wire, the size, and therefore weight, of which depends upon the amount of current to be transmitted, the voltage of transmission, and the drop or loss to be allowed. On lines up to 50 miles the loss may be economically confined to 10 per cent.

Diagram 16 gives quantity of copper wire for transmission line, for different output and voltage, at 5 per cent. line drop, per mile, to which are to be added 50 poles, 100 cross-arms, 150 pins and insulators, for single circuit 3-phase line.

Transformers to raise and lower voltage at terminals of line cost $6.00 per kilowatt of output.

A *substation* has to be provided at market end of line, which may be estimated for at $1.00 per horse-power.

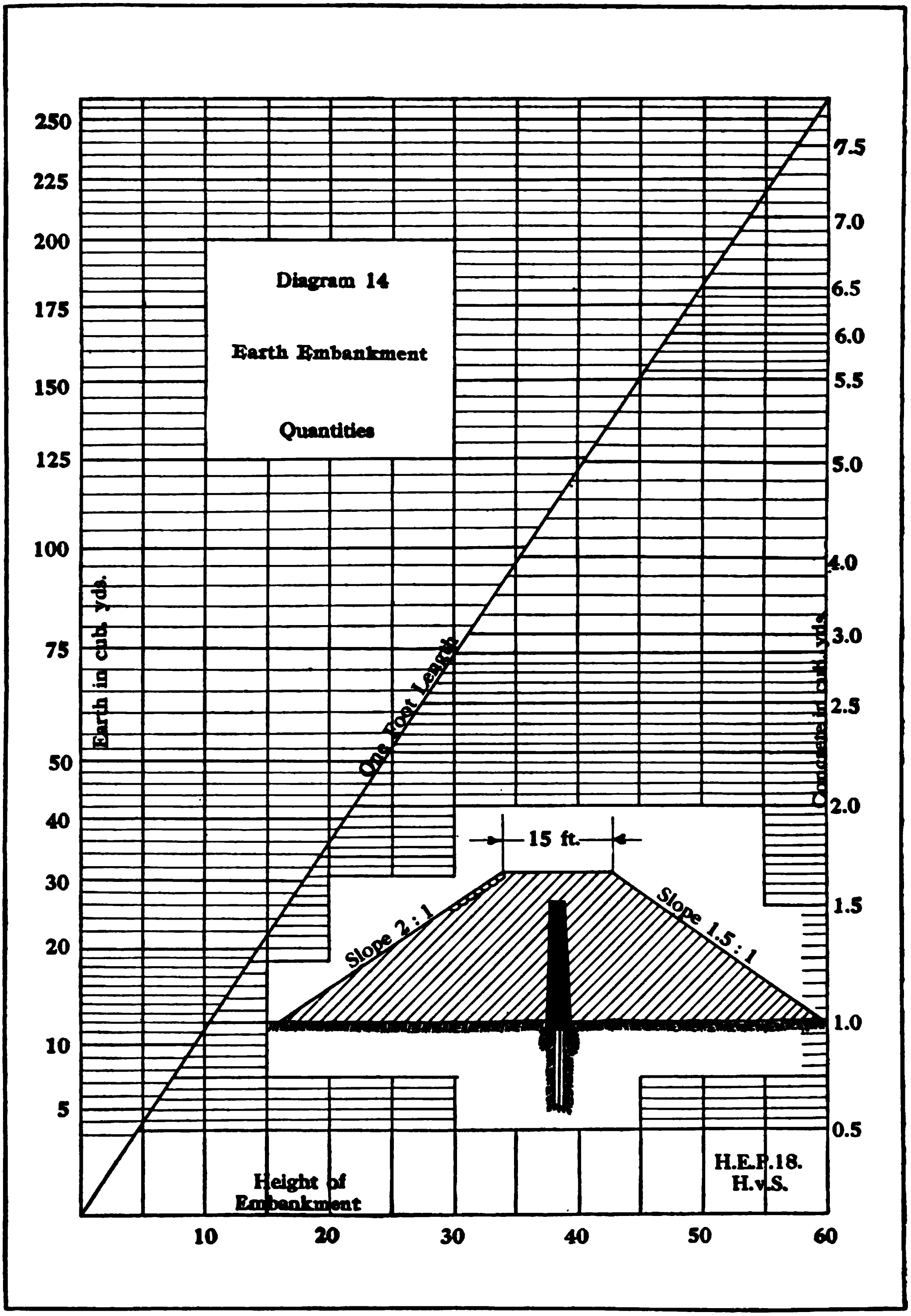
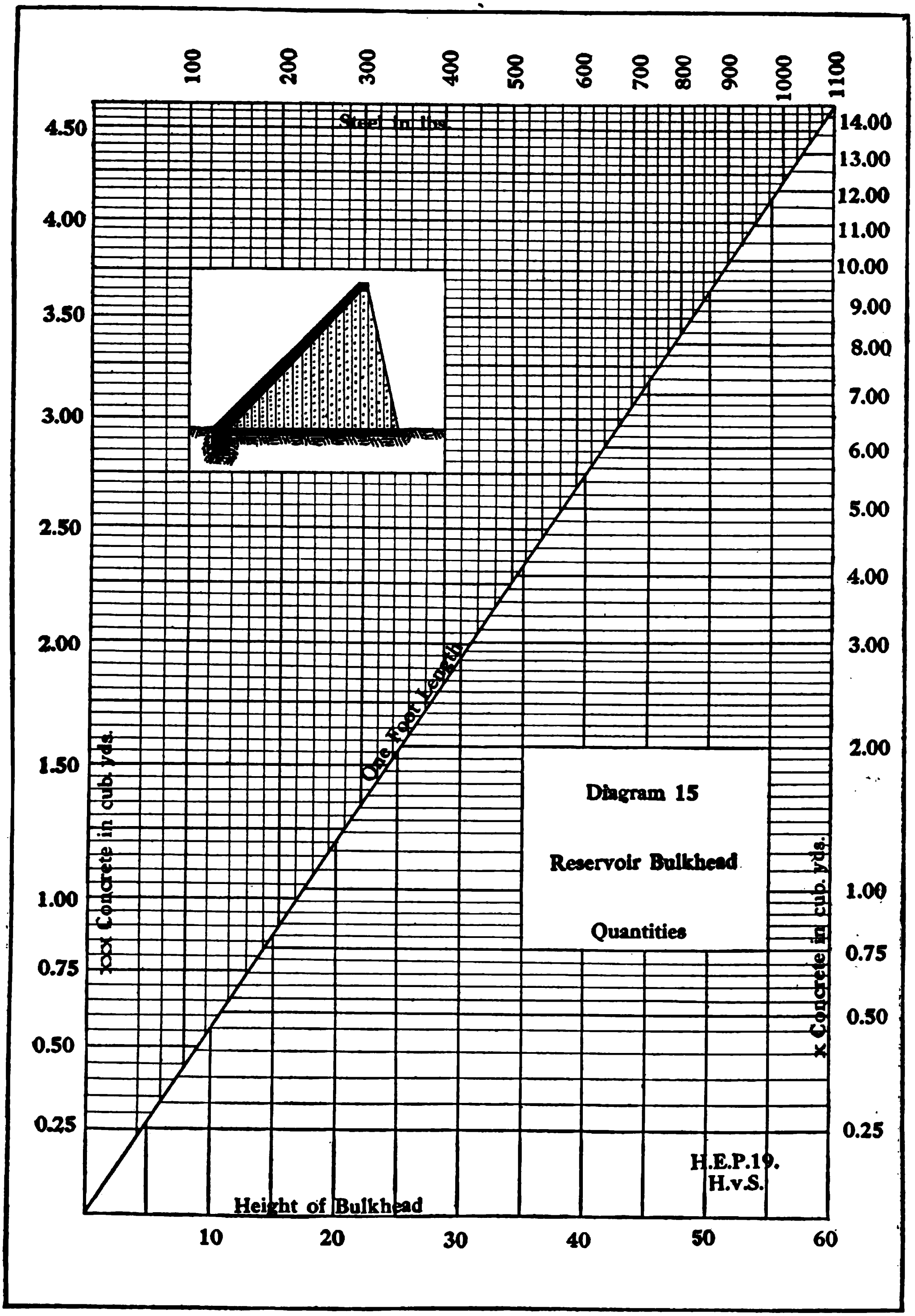
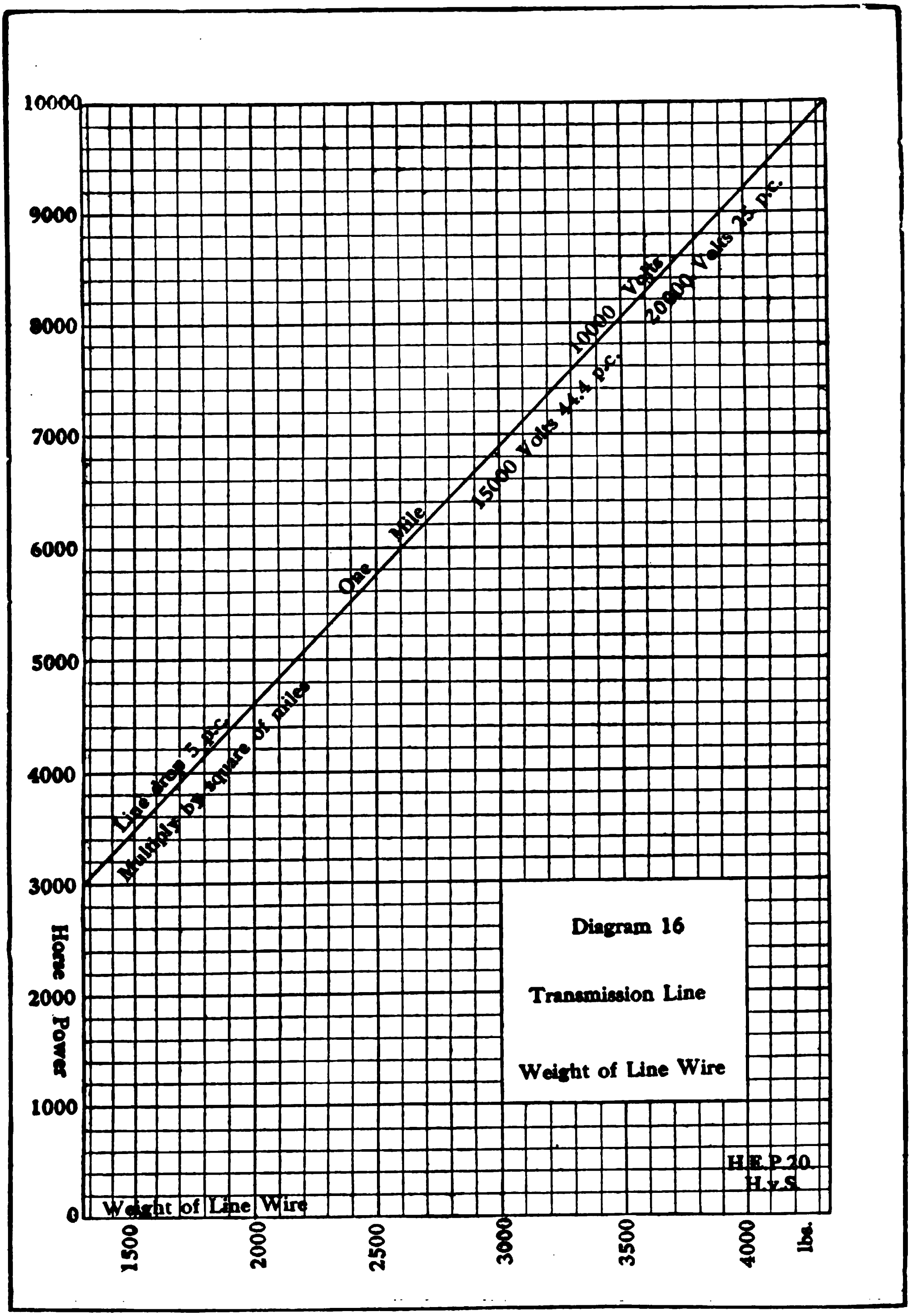


Diagram 14
Earth Embankment
Quantities
Earth in cub. yds.
Concrete in cub. yds.
One Foot Length
15 ft.
Slope 2 : 1
Slope 1.5 : 1
Height of Embankment
H.E.P.18.
H.v.S.
250
225
200
175
150
125
100
75
50
40
30
20
10
5
7.5
7.0
6.5
6.0
5.5
5.0
4.0
3.0
2.5
2.0
1.5
1.0
0.5
10
20
30
40
50
60


Steel in lbs.
100
200
300
400
500
600
700
800
900
1000
1100
4.50
4.00
3.50
3.00
2.50
2.00
1.50
1.00
0.75
0.50
0.25
xx Concrete in cub. yds.
14.00
13.00
12.00
11.00
10.00
9.00
8.00
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.75
0.50
0.25
x Concrete in cub. yds.
One Foot Length
Diagram 15
Reservoir Bulkhead
Quantities
H.E.P.19.
H.v.S.
Height of Bulkhead
10
20
30
40
50
60


10000
9000
8000
7000
6000
5000
4000
3000
2000
1000
0
Horse Power
Line drop 5 p.c.
Multiply by square of miles
One Mile
15000 Volts 44.4 p.c.
10000 Volts
20000 Volts 25 p.c.
Weight of Line Wire
1500
2000
2500
3000
3500
4000
lbs.
H.E.P.20.
H.v.S.
Diagram 16
Transmission Line
Weight of Line Wire

ARTICLE 29.—The probable *cost* of the development may be compiled from these approximations by finding cost of dam from Diagrams 9, 10, 11, and 12, cost of diversion works from a location profile, cost of power-house from Diagrams 11 and 13, cost of reservoir structures from Diagrams 11 and 15, taking cost of power equipment at $20.00 per horsepower, cost of transmission line from Diagram 16, and cost of transformers and substation as stated.

To these items must be added 10 per cent. for engineering and inspection, and to the total the value of lands, right of way, of charter and franchises.

CHAPTER V

VALUE OF PROJECT AND PRESENTATION

THE summing up of the findings from the investigations of the market, power capacity, feasibility and practicability, and of the cost of the proposed development are presented in the Engineer's report with such documentary proofs and legal opinions as the conditions call for.

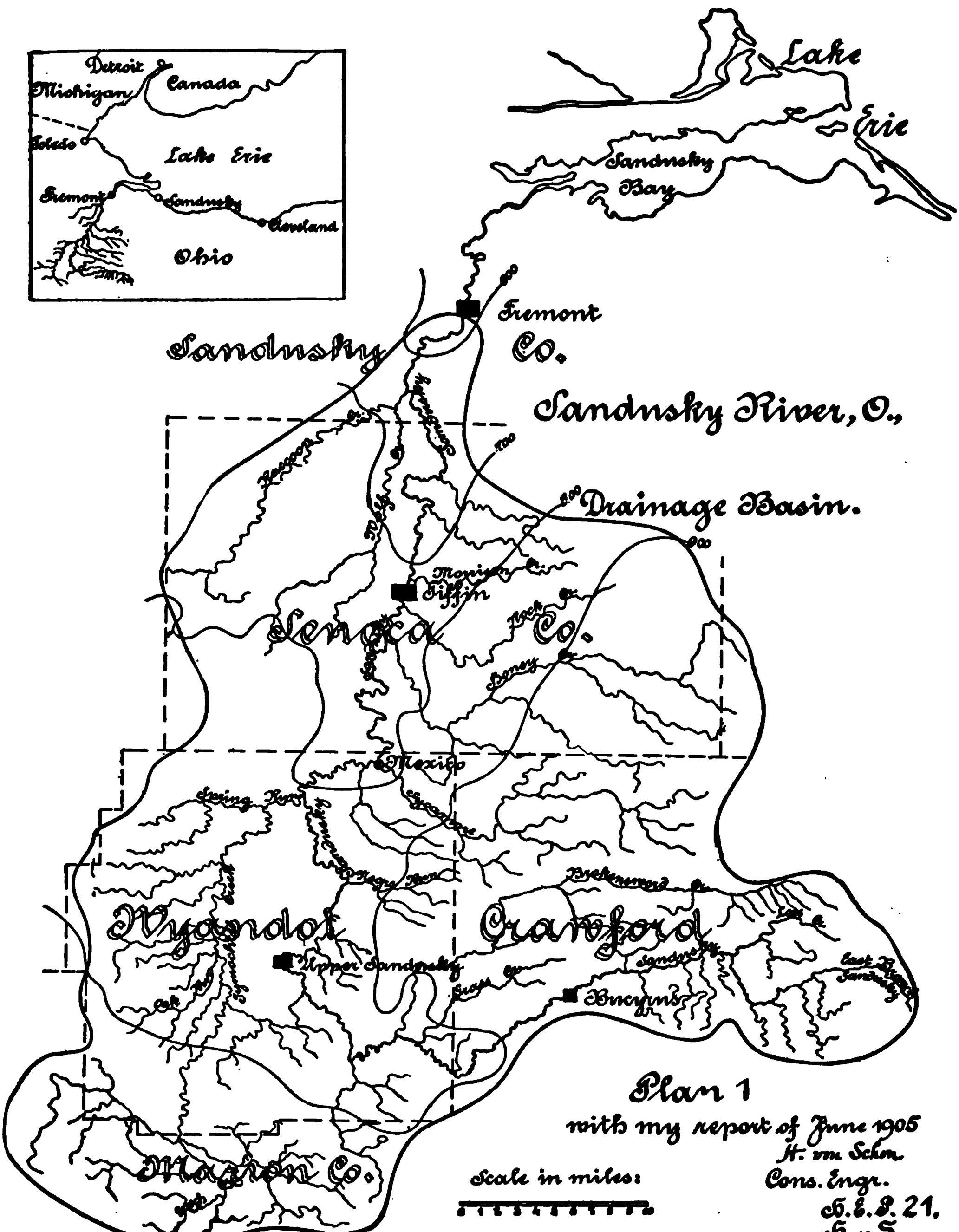
ARTICLE 30.—This report should exhaust the subject in every detail which becomes a link of the completed chain. The *market* analysis should contain the list of existing power plants and users and be of the general treatment outlined in Article 6, with the addition of any other features which may be peculiar to the case in hand. Frequently the principals who exploit the project are inclined to pass by this market topic with the argument that the proximity of such or such a large city is sufficient evidence that the current will find sale; this is a mistaken policy and generally leads to a waste of time and perhaps of funds, as this market analysis will sooner or later be demanded by solicited investors. The power opportunity must be fully treated and accompanied by detail plans such as here shown of a report made by the author in 1905.

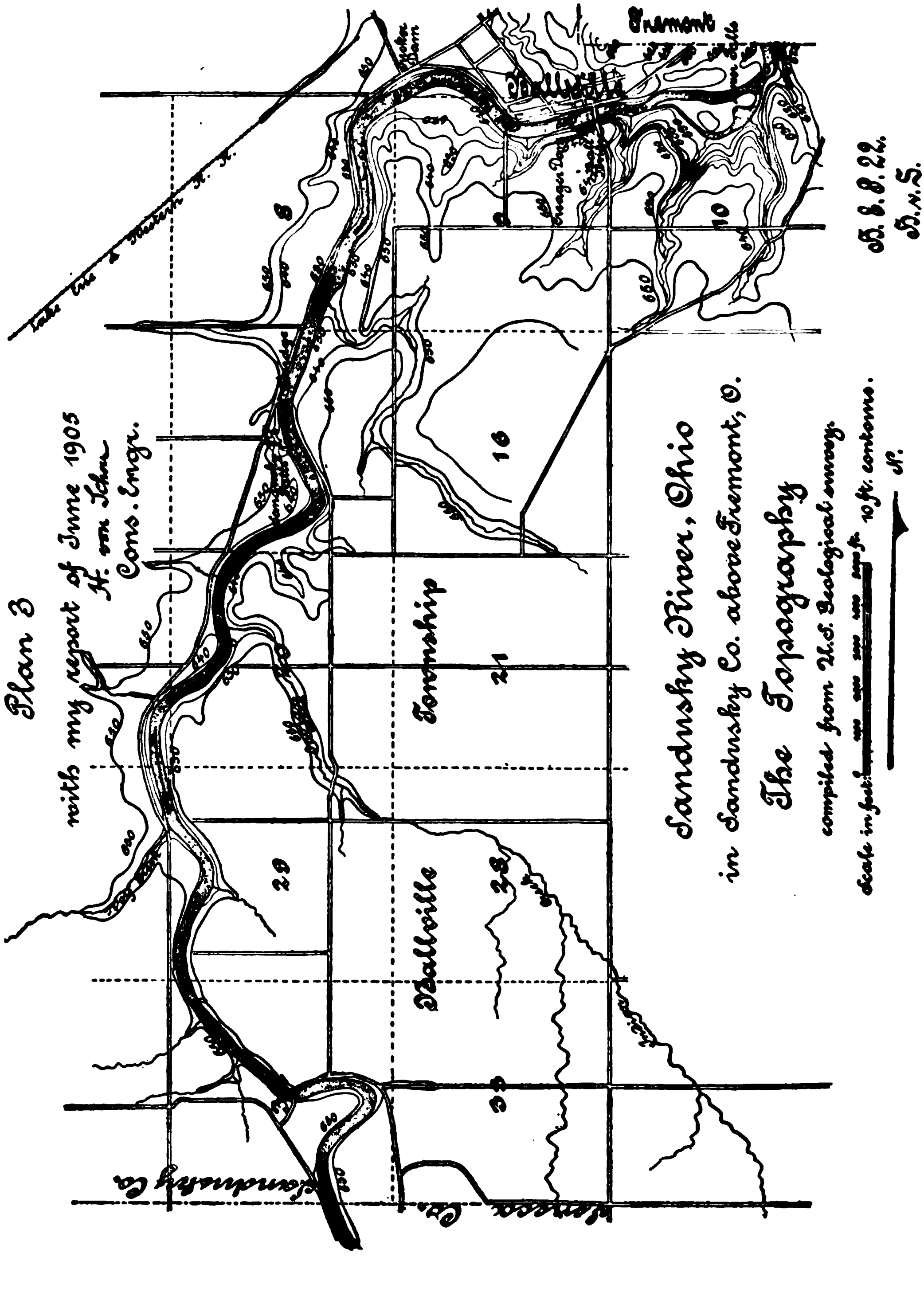
The *drainage area* should be so presented on the perspective plan that it can be readily checked, *precipitation, run-off, and flow* data considered in the determination of flow should be given in full in appendix with proper reference to the authorities from which they are taken, and every argument and conclusion as to available flow should be fully detailed.

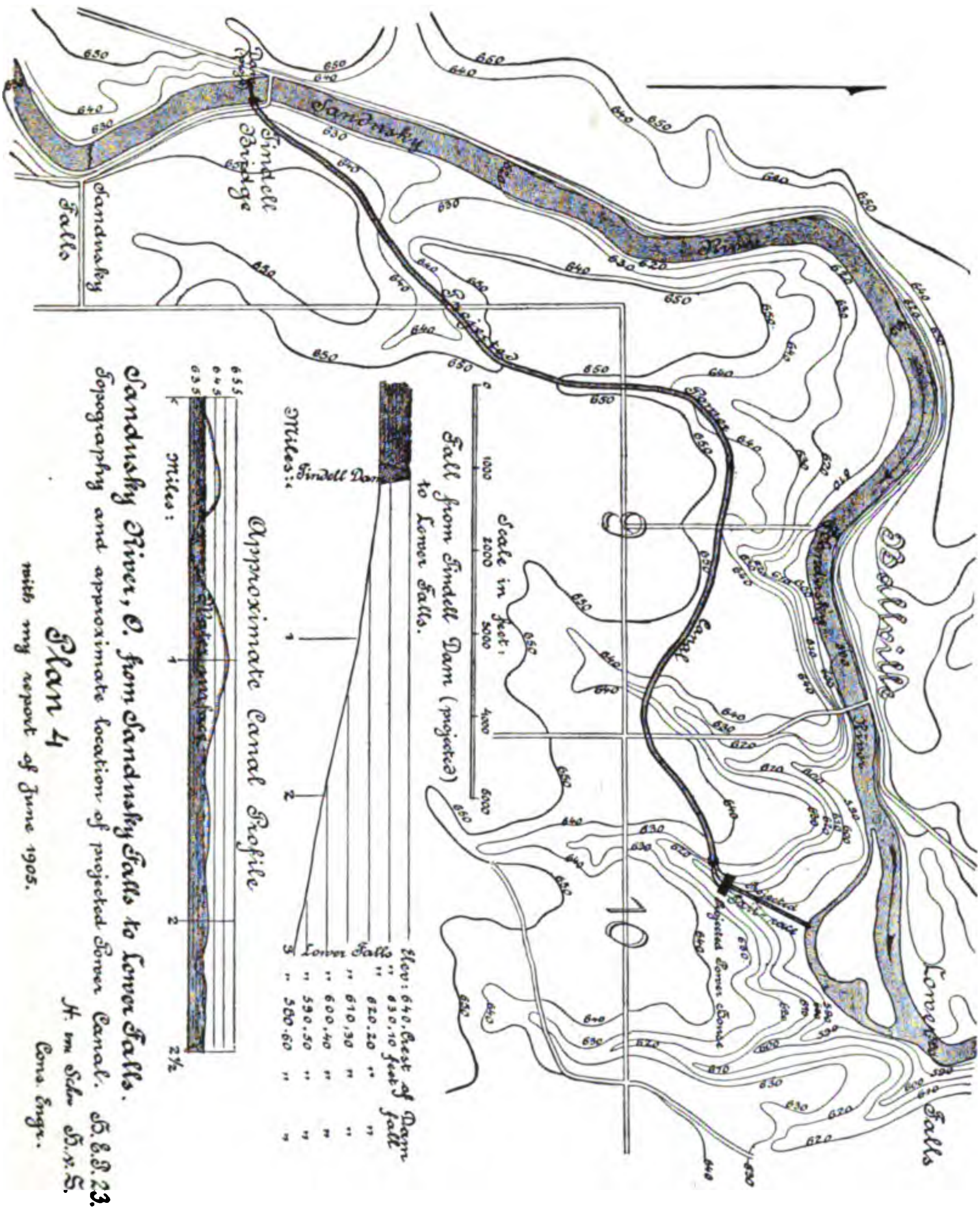
The statements as to fall should be accompanied by the detail level field notes and by profiles.

Then the *development scope* must be treated clearly, as presented in Article 51, giving a tabulation of the daily flow of the dry year, the required storage and auxiliary power supplements, and finally of the probable cost of the product for various scopes from available and supplemental sources. This last analysis can of course not be made until at the conclusion of the report, when it is followed, as will be noted later, by the detail investment balance.

The recommendable development program is then outlined and


Detroit
Michigan
Canada
Toledo
Lake Erie
Fremont
Sandusky
Cleveland
Ohio
Lake
Erie
Sandusky
Bay
Fremont
Sandusky
Co.
Sandusky River, O.,
Drainage Basin.
Tiffin
Seneca
Co.
Rock Cr.
Mexico
Wyandot
Crawford
Upper Sandusky
Bucyrus
Marion Co.
Plan 1
with my report of June 1905
H. v. Schon
Cons. Engr.
Scale in miles:


Plan 3
with my report of June 1905
Cons. Engr.
Sandusky River, Ohio
in Sandusky Co. above Fremont, O.
The Topography
compiled from U.S. Geological survey
Scale in feet
10 ft. contours
N.
Ballville
Township
Fremont
Sandusky Co.
8
9
10
16
21
29
28

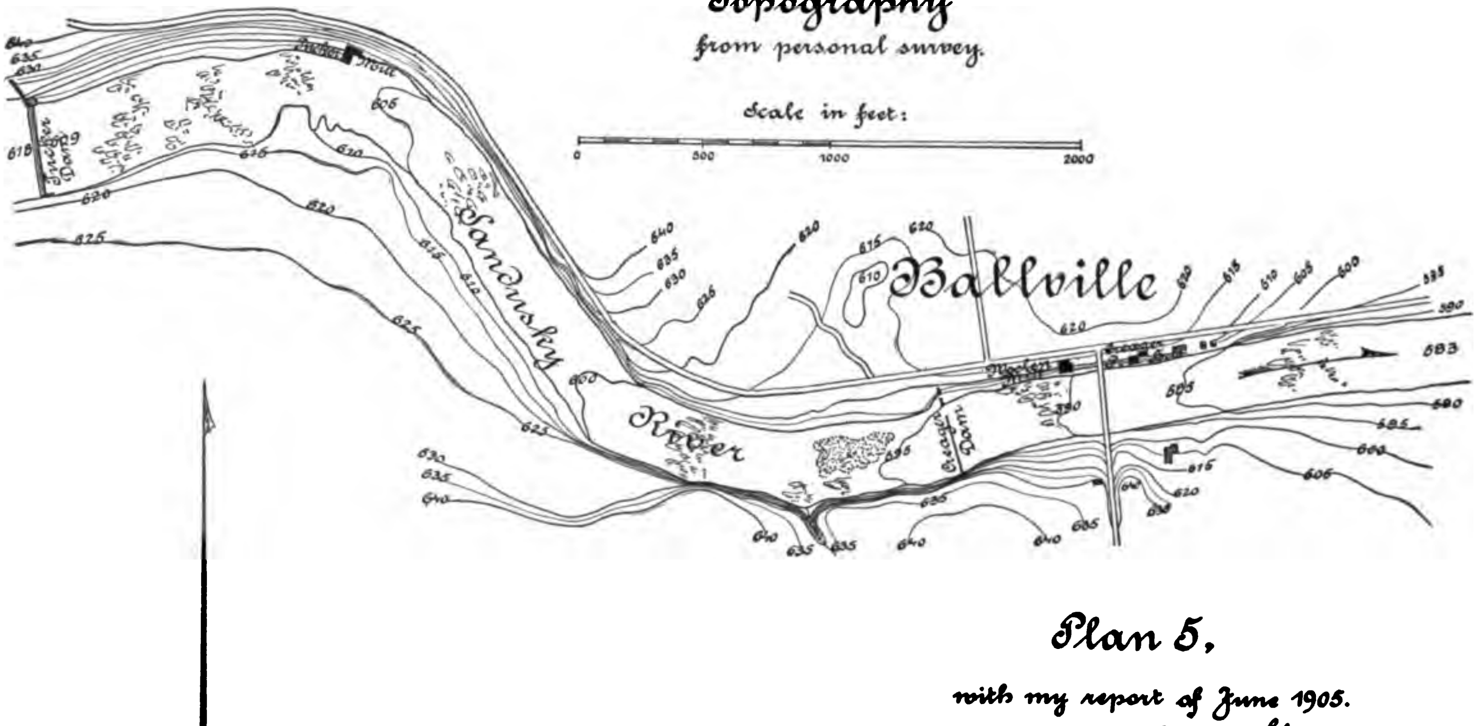

Sandusky River
Tindell Bridge
Sandusky Falls
Dellville
Lower Falls
Scale in feet:
Fall from Tindell Dam (projected) to Lower Falls.
Tindell Dam
Miles:
Elev: 640. Crest of Dam
630.10 feet fall
620.20
610.30
600.40
590.50
580.60
Lower Falls
Approximate Canal Profile
Miles:
Sandusky River, O. from Sandusky Falls to Lower Falls.
Topography and approximate location of projected Lower Canal.
Plan 4
with my report of June 1905.
Cons. Engr.

Sandusky River, O.,

at and above Ballville.

Topography

from personal survey.



Plan 5,

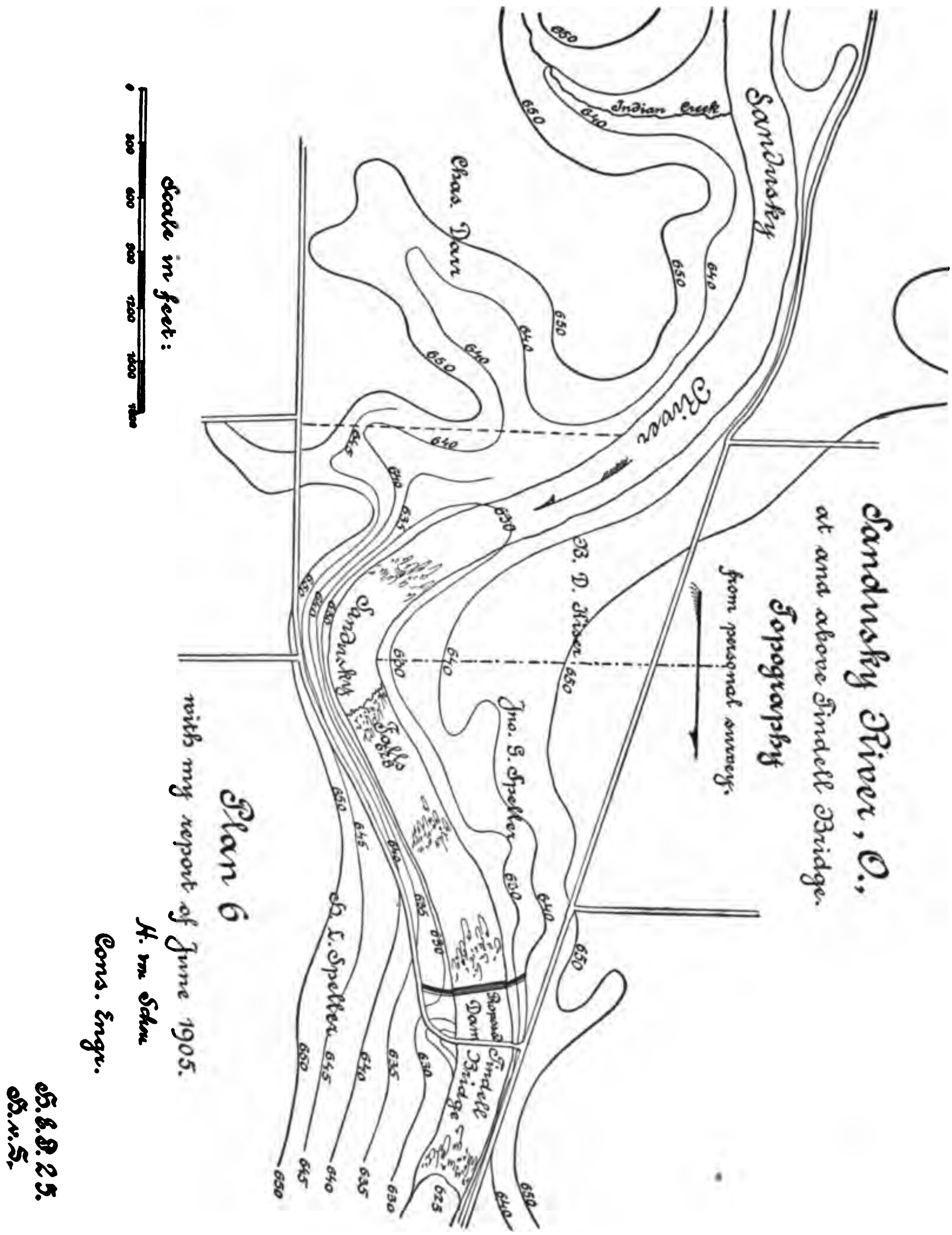
with my report of June 1905.

H. v. Schon

Cons. Engr.

B. S. P. 24.

B. v. S.


Sandusky River, O.,
at and above Tindell Bridge.
Topography
from personal survey.
Plan 6
with my report of June 1905.
Cons. Engr.
Scale in feet:
Indian Creek
Sandusky
River
Chas. Dann
B. D. Kiser
Jno. G. Speller
Falls
S. L. Speller
Proposed Dam
Tindell Bridge

illustrated by suitable plans and profiles. The author of the report should offer ample evidence by arguments, comparative cost and output data, and conclusions that the recommended program will guarantee the most resourceful and remunerative development. It is not a wise or helpful policy to suggest alternative programs, as one must be capable of being proven preferable of all.

This is followed by a general description of the structural types of the proposed works, with brief stability proofs of the essential parts and a concise specifying of especially adapted construction program and methods. Then the estimates are presented, introduced by quotations for materials as of recent date with delivery cost to site. The estimates should be in greatest detail as to every structure and its divisional construction parts, with dimensions, weights, quantities, etc., of every class of structural material, proper qualification of labor, machine and shaping costs with construction profit added. It is unwise to quote estimates in lump sums for dam, canal, power house, equipment, etc., the items involved in dam cost may be control of flow during construction by sheet piling, cribbing, diking, coffer dams, pumping, and others; preparation of river bed for foundation by excavations, trenching for cut-off wall, drilling for anchors, driving bearing or cut-off piles and more; the superstructure may consist of various parts such as base, apron, sluice ways, piers, spillway, bulkheads, abutments, etc. To lump the estimate of all of these in one item for the dam is not a satisfactory treatment of the matter in this report and will sooner or later call for itemizing, and as the lump cost cannot be correctly known to the author of the report without his having developed it step by step through these detail items, there is no reason why they should not be given in the report. The same holds good of the appurtenances of the dam, waste flume, and sluice gates, flashboards and fishladders, etc. The estimates for diversion conduit, power house, equipment, and, in fact, of every structure and part should all be segregated into the minutest details which call for material and operations. To the total cost, a proper addition must be made for engineering, inspection, insurance, and finally some arbitrary per cent. depending upon the thoroughness of the detailing of items, for incidentals which covers accidents, breaks, washouts, delays, etc.

Article 31.—The report is now complete to be closed with the investment balance, which is based upon the most remunerative scope which can now be definitely determined.

This balance must assume a total investment which is the development cost, interest payable during construction period, cost of lands, rights, franchises, and legal and engineering services.

The debits for this balance are fixed charges: interest, depreciation, administration, taxes, insurance, and operating cost, and from this will be found the cost of the product in horse-power or in kilowatt-hour for any assumed load factor.

PART II

DESIGNING AND CONSTRUCTING THE DEVELOPMENT

THIS PART treats of the engineering of the hydro-electric development; in it are presented methods, theories, designs, and their execution, as they have been found, in the author's practice of this specialty, to secure the desired results in a manner adapted to the commercial as well as the engineering requirements of the business. Some of these leave the trodden paths of former practice; the majority must needs follow them; only where necessary, in the author's judgment, to make clear his meaning, have rudimentary methods been employed; on the whole, it has been the purpose to render the treatment of this subject complete within its scope.

The subheads are of five chapters dealing with surveys, development programme, structural types, equipment, plans, and estimates and specifications, construction, and superintendence, each of which is divided into articles covering detail topics.

CHAPTER VI

THE SURVEY

SURVEY embraces all operations by which the hydrographic, topographic, and geologic characteristics are investigated and determined.

ARTICLE 32.—The first preparation for the work of surveys is the *examination of maps* of the stream and the projected power-plant location, if such has been fixed. The best obtainable maps are the United States Geological Survey topographic sheets, which may be secured from the Director of the Survey, in Washington, D. C., or at local agencies. Several of the States provide annual appropriations for co-operative surveys with the Federal department, and the annual reports of the State Engineers, or Geologists, contain topographic county maps of similar origin. This is the available information from which the topography of a stream system, or so much as is involved in the examinations,

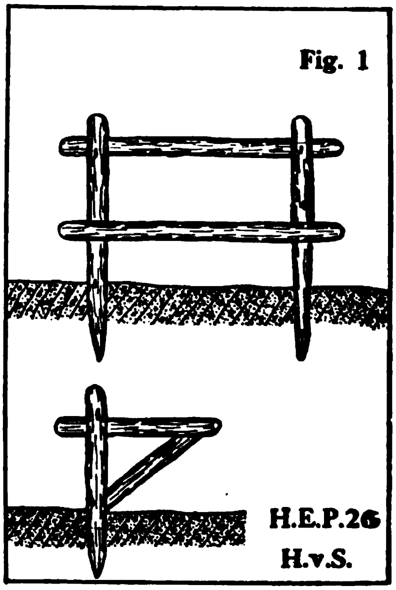
may be studied; it will reveal the course of the river, the contour formation of its banks, its fall, by the crossing of successive contours, the locations of railroads and highways, of fords and ferries, and of settlements. All this furnishes ample data for the appreciation of the general conditions, which will become useful in locating dam site, estimating fall, flowage areas, and storage opportunities.

An examination of the drainage area on State maps and of precipitation records will throw considerable light upon the probable flow characteristics of the watercourse; note the origin of its principal sources, whether in foot-hills or flowing out of swamps or lakes, and the length of its tributaries, the import of all of which will be discussed in detail further on.

Finally, much practical information as to subsurface formations in the vicinity of the projected power site can be gained from well borings; one concern frequently operates a well-boring apparatus in several counties, and these may be traced through hardware merchants at the county seat or near-by town.

ARTICLE 33. *Reconnaissance.*—With the general knowledge of hydraulic, topographic, and geologic characteristics thus gathered, a reconnaissance is the next best preparation, on horseback or preferably in a boat floating down the river, equipped with a camera, compass, hand-level, aneroid, field-glasses, sketch-book, and the plan of the river's course, showing county, township, and section lines. On such a trip many details will be revealed which are not on the maps or are given erroneously. The aneroid should be read at regular intervals of time, the location being identified on the map; the velocity of flow, and therefore of transit, can be estimated and thus distances sufficiently determined to aid in fixing the fall. Observe the character of the banks and make notes in the sketch-book of likely dam sites, estimating the river's width and finding the approximate and relative heights of the banks by aneroid and hand-level; note also the improvements on river bottom lands and the height of bridge crossings, paralleling railroads and highways. A few days devoted to reconnaissance will prove an exceedingly valuable investment, the benefits of which will be frequently recognized in the course of perfecting the development programme; above all, it is highly recommendable to make copious notes and sketches of whatever is worth remembering and to take photographic views of the most notable features.

Article 34. *Triangulation.*—With this general equipment the work of definite determinations can be approached. It is not always feasible to foretell what the extent of the survey will be. It must cover the dam site, the flowage area, and perhaps the tracing of property lines. It is, however, always advisable first to establish some fixed references for elevations and points by a system of *triangulation* of one or more quadrilaterals; this should be planned to be readily accessible and safe from interference during construction and above the highest pond level. The base should be at least twice as long as the width of the stream valley. The line is instrumentally projected and permanent base-point markers of stout posts or of stones are set. Measuring benches (Fig. 1) are placed at intervals of 100 feet along the line; they consist of two stout vertical stakes set twelve inches centres and a horizontal piece with two-inch flattened top face secured to them; the bench pieces of each successive 100-foot section are on same level, and where this changes two bench pieces are secured to stakes. Supporting brackets (Fig. 1), consisting of a stake with one horizontal piece at the level of the respective 100-foot section, are set 25, 50, and 75 feet from the section bench. The measurement should be made with a standardized 100-foot steel tape, which is placed on supporting brackets, ends on benches, and a ten-pound weight secured to each handle; the zero is marked on the permanent base point and the fractional foot to the marked centre of the bench piece is measured with a hardwood scale to one-hundredth of a foot; record of the length of each section of base is kept. This measurement should be repeated three times and the mean accepted. The selected triangulation points are permanently marked, and *tripods* (Fig. 2), constructed of fence-posts, are placed securely over them, the legs being set three feet in the ground; tops are covered by a two-inch wooden plate with a two-inch hole in its centre and plumbed over the base point; the top of the tripod should be from four to four and a half feet above the ground. Triangu-



Base Benches; Base Supporting Brackets.

lation points are preferably marked with *targets*, consisting of two boards one inch thick, twelve inches wide, and three feet long, secured to each other as shown in Fig. 2, wings being painted alternately white and red; a two-inch handle, secured to the end, is set into the hole of the tripod plate.

The angles should be measured with an engineer's transit reading by vernier to thirty seconds. The instrument is secured to a *trivet plate* with three spikes which rest on the tripod plate. Each angle should be measured five times in both directions, giving ten readings; the mean is accepted.

Fig. 2

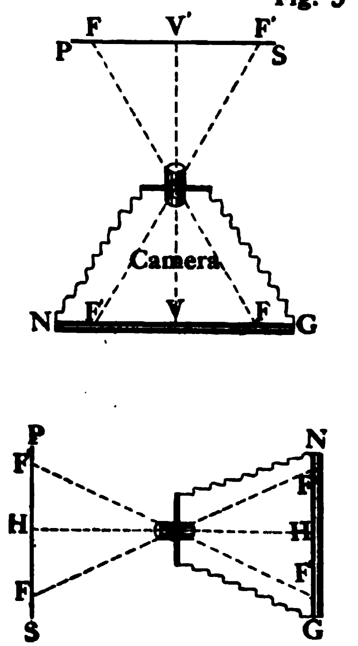
Tripod and Target.

The *azimuth* of the base should be determined from Polaris, so that any station in the survey system may be available for the tracing of the magnetic bearings given in the boundary descriptions. The sides of triangles are computed by trigonometrical functions; the location of points is determined by co-ordinates.

ARTICLE 35. *Elevations.*—A *reference bench* is selected or established, and the elevations of the triangulation points are determined by return levels from reference or line-benches and plainly marked on the station tripod. An eighteen-inch "Y" level in good adjustment and rods reading to hundredths of a foot are used in running the level lines. Triangulation benches being established, a level line is run up and down the stream over the entire reach affected by the development. When the immediate river shore is inaccessible or the stream tortuous, the line may follow along the top of the bank, or at some distance from it along roads, and *levels taken to water* at accessible points. Every fifth turning-point should be a bench of permanent mark, and each section of level line between benches should be returned until an agreement between two runs within 0.001 foot is secured; where convenient the line benches should be connected with the triangulation benches for check. Elevations are plainly marked on all permanent benches.

ARTICLE 36. *Topography.*—A *stadia survey* is made of all the river valley to be considered in the development; it is referred to the triangulation system. This survey should develop the topography to one-foot contours at the probable location of the works, and in five-foot contours over the remaining territory; the bearings should be of azimuths in harmony with the triangulation system; distances are read on plumbed rods; vertical angles are measured to the height of instrument; stadia stations are marked by stakes with tack, and the instrument is plumbed. Whenever practicable, reference readings are taken to triangulation and water points. By this survey, with a rodman on each side of the river, the shore lines, property corners and boundaries, buildings, bridges, roads, and a sufficient number of contour points to project the true topography are located. Plan 7 shows the *results* of triangulation, levelling, and topographic *survey.*

Fig. 3



H.E.P.28
H.v.S.

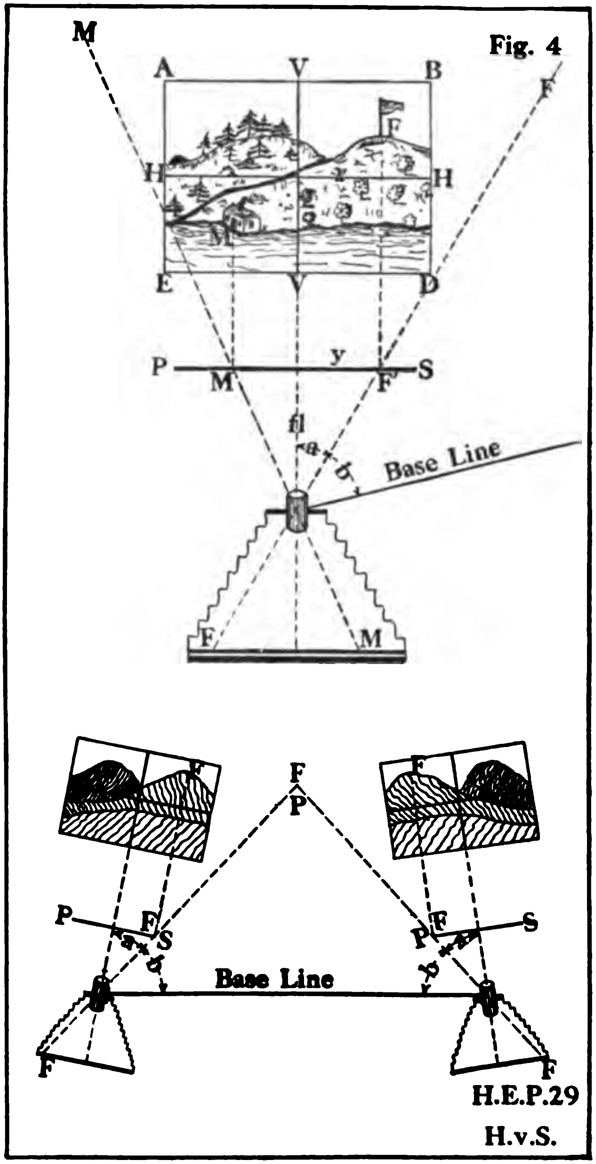
The instrument's adjustments should be checked at the beginning and close of each day's survey.

ARTICLE 37. *Phototopography.*—It is not the purpose to enter upon a broad discussion of the subject of photogrammetry, but to describe only the practical process by which a general projection of the topography of the stream valley and its immediate high banks can be obtained within contour intervals of about ten feet.

In a photographic camera (Fig. 3) C is the optical centre, being the middle point of the optical axis between the two lenses of the objective; N G is the negative plane in which the sensitive plate or film rests; V V′ is the optical axis produced and its vertical projection called the vertical; H H′ is the horizontal projection of the same axis known as the horizon; C V is the focal length which is uniform for distances beyond 100 feet; P S is the imaginary positive plane, being parallel to the negative plane and focal length from the optical centre.

The image on the negative plane is the reverse of the object it repre-

sents and in true perspective, but if reflected on the positive plane it would there appear as a perfect miniature facsimile; the points F and F′ in the negative plane would be found as F and F′ in the positive at like distances horizontally or vertically from the vertical or horizon.



This is the optical and geometrical principle on which the utilization of the camera for this purpose is based.

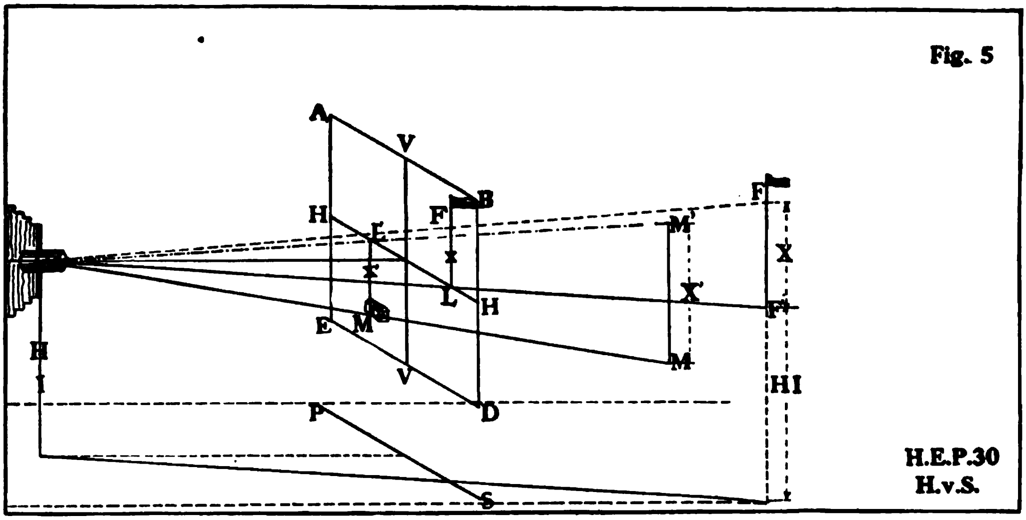
Fig. 4, A B D E, is a print of the negative laid down flat with H H′, the reproduced horizon, parallel to P S, the positive plane, and at any convenient distance above it; V V′ is the vertical produced; F is the image of a flag on top of a hill. *To locate F graphically* drop a perpendicular from F to the positive plane at F′ and connect this point with the optical centre C, fixed in V V′ produced and focal length, in natural scale, from the positive plane. The true location of F will be in the extension of the line C F′, and will be fixed by a similar operation with the second view taken from the other end of the base line, as the two projections will intersect at the location of F in the plan.

To find the location of F algebraically measure "y" on the print, being the distance along the horizon of the vertical projection of F to

the image vertical, then tg. a $= \frac{y}{fl}$ (fl = C V focal length) angle a + b is known from azimuths of triangulation points, so is the length of the base line C C′ and therefore also lines C F and C′ F which are plotted in the scale of the base line.

In this manner all points which appear on two views can be located.

To find the Elevation of an Object Algebraically.—Fig. 5, A B D E, is the positive plane; F is the flag the elevation of which is to be determined. Drop the perpendicular F L to the horizon and connect L with C, the optical centre, measure F L in natural scale from the point; then



C L (focal length in natural scale) : F L = C F′ : F F′ ; C F′ has been found as per preceding discussion and is expressed in the scale of the base line, and F F is the elevation of the flag point above the horizon expressed in the scale of the base line. This determination can be repeated from the other view containing F, thus affording a valuable check. For F the elevation is above horizon and therefore to be added to H I, height of instrument or its elevation; for M, being below the horizon, the reverse is the case.

To find the Elevation Graphically.—Fig. 6, A B D E, is the positive print, F the image of the flag, P S the positive plane, C N the focal length; drop a vertical from F to the positive plane in F′, connect C with F′ and produce to F as per previous discussion in the scale of the base line; measure x, the height of the flag point above the horizon, on the print,

draw a perpendicular to C F at F equal to x in the scale of the print, then $CF' : x = CF : x'$, which is the height of the flag point above the horizon measured in the scale of the base line.

The practical operating programme is as follows:

The camera is attached to an engineer's transit by means of screw hooks fixed on the side of one of the standards, and two lugs or sleeves correspondingly spaced are secured to the side of the camera box. When the camera is thus suspended from the transit standard and the latter is levelled up, the following conditions are met:

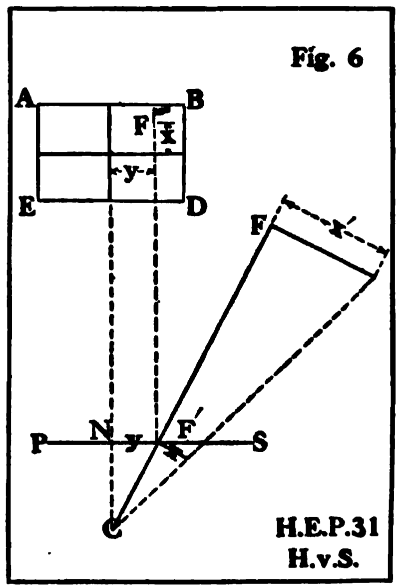


Fig. 6

1. The optical centre of the camera objective lies in a vertical plane over the station point occupied by the transit.
2. The optical camera axis is in a vertical plane parallel to the optical transit telescope axis.
3. The optical camera axis lies in a horizontal plane with the transit telescope revolving axis.
4. The negative plane is vertical.

These four conditions remain rigid throughout the operations here considered—that is, the more complex and broader practice of photogrammetry in which the negative plane may be inclined from the vertical or of curved surfaces does not here enter.

Any commercial camera can be employed with satisfactory results, the lighter the better; a 5 x 7 is well adapted; the objective should be rectilinear and the focal length uniform for distant points.

The vertical and horizon are marked on the ground glass or finder, and small arrows or pointers are fixed to the interior of the four sides of the plate or film holder at the intersections of vertical and horizon with the frame, so that each view taken bears these marks, by which the control lines can be correctly reproduced on the positive print. The apparatus is now ready for operations,—that is, the survey of a stream valley, a triangulation system having been established.

5. The instrument is set over a triangulation station, the camera attached, and the transit levelled.

6. A pointing is made with the transit and known azimuth to the triangulation target on the opposite side of the stream valley.
7. The plate or film is exposed, removed, and the camera is recharged.

Sweeping the horizon in the direction of the operating programme, up or down stream, one pointing is made and view taken to each visible opposite triangulation target.

8. A record is kept as in surveys, views being identified as 1 to 2, 1 to 4, 1 to 6, etc., and the same notation is made on the plate holders as they are taken from the camera.

The same programme is repeated from the opposite side,—that is, 2–1, 2–3, 2–5, etc.

The operator needs no technical photographic knowledge or skill beyond adjusting, charging, and unloading the camera, protecting plates from being light-struck, guarding against halation,—that is, exposures toward the sun,—and a practical guide to the use of the proper diaphragm or stop and the length of exposure under different conditions of light and time of day. Such an exposure and diaphragm scale should be adapted to the characteristics of the objective and the sensitiveness of the plate, and the data required can readily be obtained from the photographic supply house where these are secured.

Development of the exposed plates or films and the printing of the positives had best be delegated to a photographer. In practice it will be found advisable to duplicate all views, to make certain by examination of image on ground glass or in finder that the key point connecting with the previous view is in the field of view. The instrument must be examined before each exposure as to its level condition, as the weight of the camera may throw it out. The prints should be strong, and in order to secure detail an orange-colored screen may be fixed in the camera immediately back of the objective.

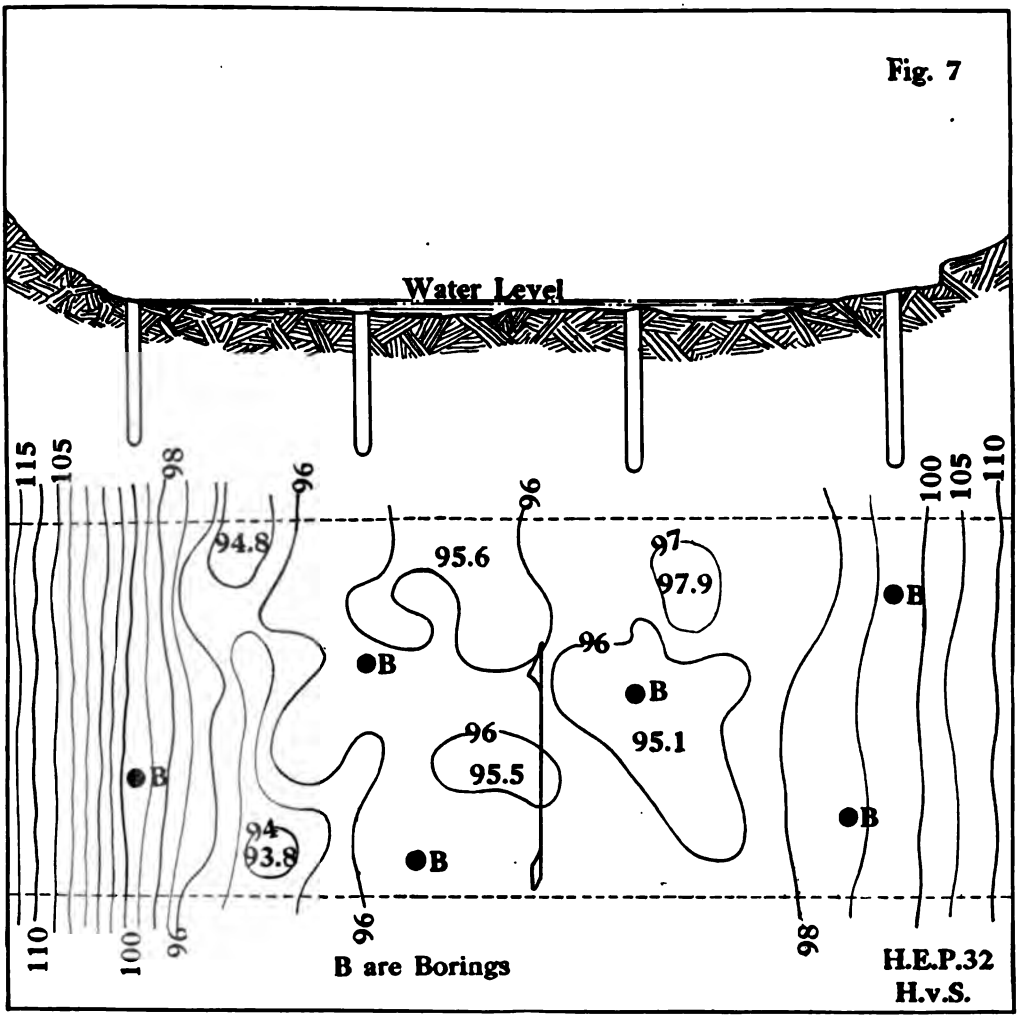
9. For the projection of the plan the horizon and vertical are drawn on the prints in vermilion, and likewise all objects to be plotted are marked by small circles or dots of the same color and numbered identically on the different prints containing them.

After the principal points are projected and their elevations have been determined, the contours and topographic characteristics are plotted on the plan as indicated in the prints.

As stated at the beginning of this subject, the results to be expected

are of a general character only, and therefore the refinements of guarding against change of paper texture have no place here.

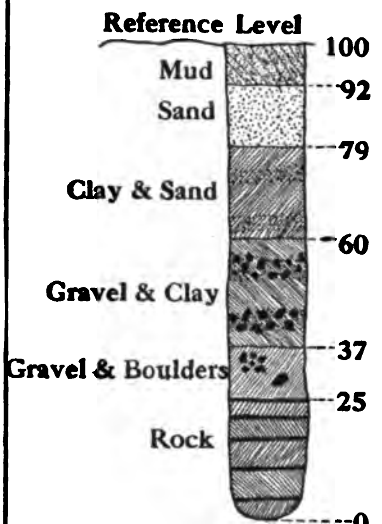
ARTICLE 38.—*Detail surveys* should be made of gauging sections, recommendable dam sites, and of the location of the diversion works;



they are referred to bench marks by determination of land or river bed and of water surface elevations at intervals of five feet by means of "Y" level, the points in the river being taken from a boat passing across along a line held taut by means of upstream guy ropes, the line being suitably marked off in five-foot lengths. One such section suffices at the gauging point, the terminals being fixed by posts; gaugings of different days

should be preceded by re-sectioning in order to detect changes in the river bed. At the dam locations a section of the stream, two hundred feet long, should be covered by transverse lines five feet centres; canal, flume, and pipe line locations must be cross sectioned every ten feet longitudinally for a width double that of the probable construction, the centre line being traversed and marked at 100-foot points. The power-house location site should be cross sectioned as described for the dam site.

Fig. 8



H.E.P.33
H.v.S.

Fig. 7 shows the plan and cross-section of a dam site.

ARTICLE 39.—*Borings* should be made on sites of dam, diversion works, and power station, being carried to rock or impermeable material, and about fifty feet centres in each direction. The elevation, depth, and character of each class of material should be ascertained. The stratifications of rock for upper ten feet of its depth must be ascertained in order to discover possible water channels, nor must the rock surface found at fifty-feet points be accepted as being uniformly level between them, but intermediate borings should be made to establish fully these conditions. Gravel, clay, and sand should be classified as to their characteristics, samples of all being preserved for future reference. These investigations by borings cannot be made too exhaustively; the more thorough the knowledge of the subsurface formations, the better for the sake of safety and economy. A well-digger's outfit will prove the best for all purposes; nothing of much value can be secured with hand-boring apparatus. All borings should be instrumentally located and all elevations referred to bench marks.

Fig. 8 shows the record of a boring station.

ARTICLE 40.—*Stream gaugings* should be made daily for as long a period as practicable, which will employ a separate force of three or four men constantly. A well-conditioned section is selected, preferably on a straight reach of the river, the best obtainable being a bridge cross-

ing, and in its absence a point where the perimeter is of nearly uniform elliptical shape and the flow is not influenced by islands, shoals, rocks, or other obstructions. A gauge is set at each shore point, preferably in a recess of the shore where it will be guarded against floatage; it consists of a board graduated to feet and tenths, which is secured to a post or pile firmly set, or to a bridge pier. The gauge boards are marked to correspond with the elevation reference of the survey. The cross-section is determined as described in Art. 37. Gauges are read and a velocity measurement is made with a current meter. The programme will depend upon conveniences at hand, most readily from a flat boat passing across by aid of a ferry rope or, better, from two such boats secured together by a timber platform. The meter must be rated before and after the operation, which is most conveniently done by having two meters, one of which is used only as the standard, all the measurements being made with the other. Velocity measurements should be made at depth intervals of two feet; if the river stage fluctuates during measurements, they should be rejected. The meter is used from the upstream end of the boat; three separate readings of one, one and a half, and two minutes' duration should be taken for each observation, and the mean accepted; the boat must be held firmly in a fixed position during the observations. The measurements at all the verticals of one meter station are to be made before moving to the next station in the section. The first measurement should be taken two feet below the surface; the last, two feet above the river-bed, and at least three on each vertical. In great depths, or swift currents, a sufficient weight must be suspended from the meter to maintain it in a vertical position. Weeds and floating sand will interfere with correct measurements.

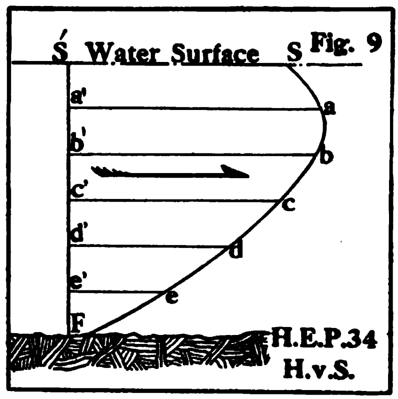


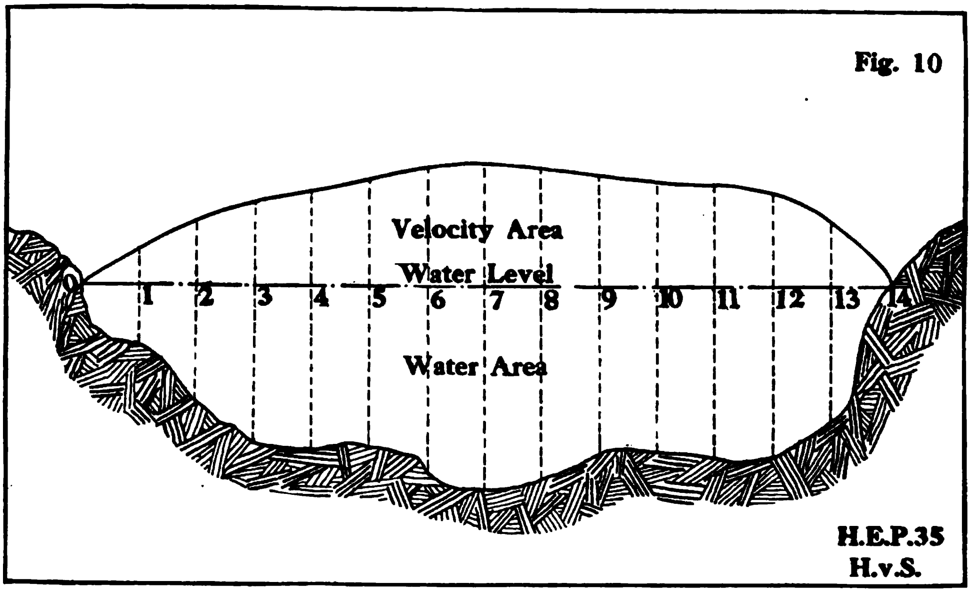
Fig. 9

The total discharge is ascertained by plotting the velocities found for each vertical as shown in Fig. 9, where S′ F represents the verticals and total depth, a′, b′, c′, d′, e′ the meter points 2 feet centres, and a′ a, b′ b, c′ c, d′ d, and e′ e the corrected velocities for each; therefore the discharge through section S′ F is represented by the area S′ S a b c d e F.

Fig. 10 represents the entire cross-section; 1, 2, 3, 4, etc. are section points ten feet centres; the lower ordinates express the depth, the upper the mean velocities; the discharge at a section point = d x v, and the total discharge is the product of the sum of all point discharges into ten, their spacing.

Meters should be run for a short time before beginning the operations, —*i.e.*, before readings are taken,—as they are likely to overspeed at first.

When current meters are not available, the velocity may be found by aid of *floats*. The gauging station is arranged as described in Art. 16;



surface floats, preferably corked bottles sufficiently weighted to project only with their necks, are placed in the water fifty feet above the up-stream gauging section limit, being the "dead run," the locus of their passage into the gauging section area is noted by line markers, and the time of such entry is taken by a stop-watch; one satisfactory run should pass under each marker; the bottles may be recovered by a boat below section. *Subsurface floats*, or double floats, are sometimes employed; they consist of a surface float from which a lower float is suspended, the theory being that thus the velocity of lower strata will be indicated; in practice this result is not readily realized.

Rod-floats are likely to secure a much more reliable measurement of mean velocity of the stream. They are one-inch square soft wood sticks

weighted at one end with lead strips or wire to float upright and as near the bottom of the river as practicable without striking it; they are set adrift fifty feet above the section, being located and timed as are surface floats; one should be sent under each line marker.

ARTICLE 41. *Reduction of Stream Gaugings.*—The discharge of a watercourse is the product of its area and the velocity of flow; the former having been found from cross-section and the latter by any of the described methods.

When surface floats are employed, the observed velocity is the surface velocity, which is reduced to the mean velocity and then plotted, as in Fig. 10, for the respective vertical. The ratio of surface to mean velocity is fixed by the depth and the coefficient of the perimeter roughness, "N," which for alluvial stream beds is—

.017 for a smooth channel bed without any obstructions to flow, such as boulders, snags, or large gravel in bed;
.020 for smooth channels with large gravel in bed;
.0225 for slightly irregular channel beds;
.025 for an irregularly contoured bed with some boulders;
.0275 for same as last with boulders and some weeds;
.030 for rough rock channel beds;
.035 for very rough rock beds with many boulders.

With these values of "N" for the respective channels, the compiled results of many experiments suggest, as an approximate ratio of mean to surface velocities, the values given on Diagram 17; when the mean velocity on a vertical has thus been found from the observed surface velocity it is plotted as shown on Fig. 10.

The velocities of rod floats represent the mean velocity of a vertical section when their submerged length is 0.99 of the depth of the water; but it is impracticable to float them of such length, and their velocities are therefore in excess of the mean velocity; the necessary corrections, as established by experiments, are given on Diagram 18.

ARTICLE 42. *Stream Discharge Curve.*—When the river's discharge is known for a considerable range of its stages, a discharge curve is projected, as shown on Diagram 19, by which the flow, corresponding to any gauge height, can be found; only constant repetitions of stream measurements, especially during the extreme low and high stages, will furnish the complete data for a reliable rating of its flow.

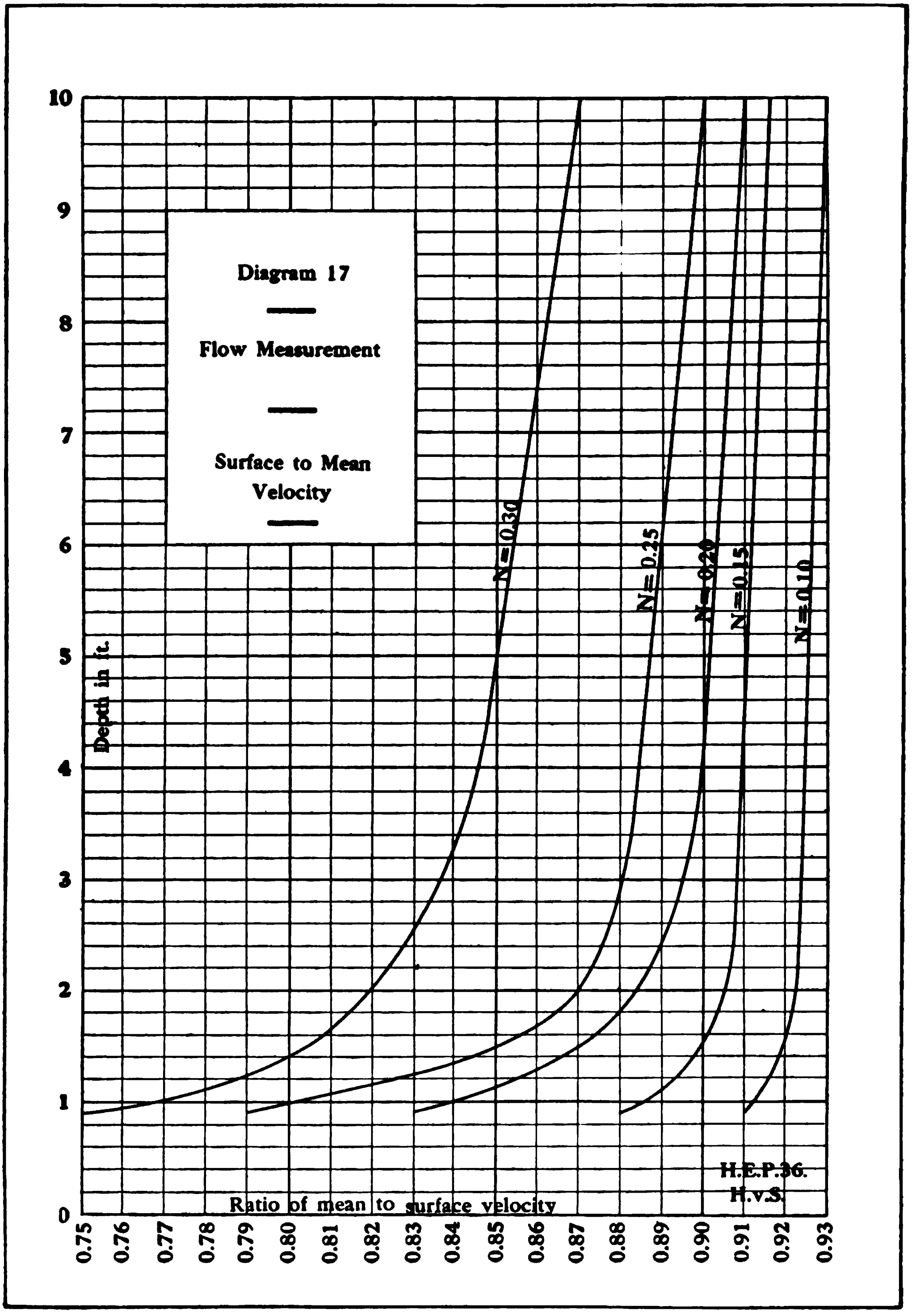


Diagram 17
Flow Measurement
Surface to Mean Velocity
Depth in ft.
Ratio of mean to surface velocity
N=0.30
N=0.25
N=0.20
N=0.15
N=0.10
H.E.P.36.
H.v.S.
0
1
2
3
4
5
6
7
8
9
10
0.75
0.76
0.77
0.78
0.79
0.80
0.81
0.82
0.83
0.84
0.85
0.86
0.87
0.88
0.89
0.90
0.91
0.92
0.93

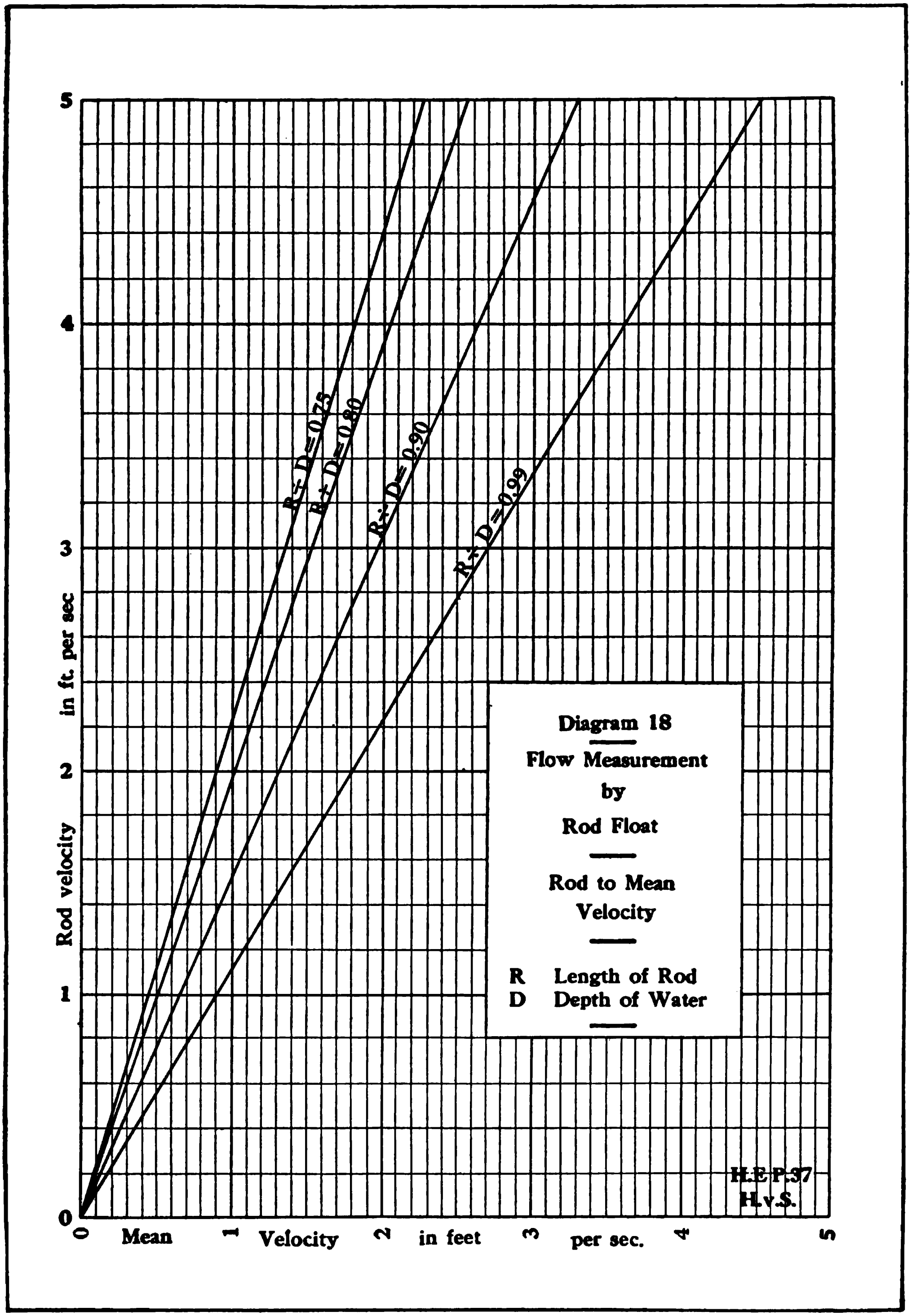


Diagram 18
Flow Measurement
by
Rod Float
Rod to Mean
Velocity
R Length of Rod
D Depth of Water
R ÷ D = 0.75
R ÷ D = 0.80
R ÷ D = 0.90
R ÷ D = 0.99
Rod velocity in ft. per sec
Mean Velocity in feet per sec.
0
1
2
3
4
5
H.E.P.37
H.v.S.

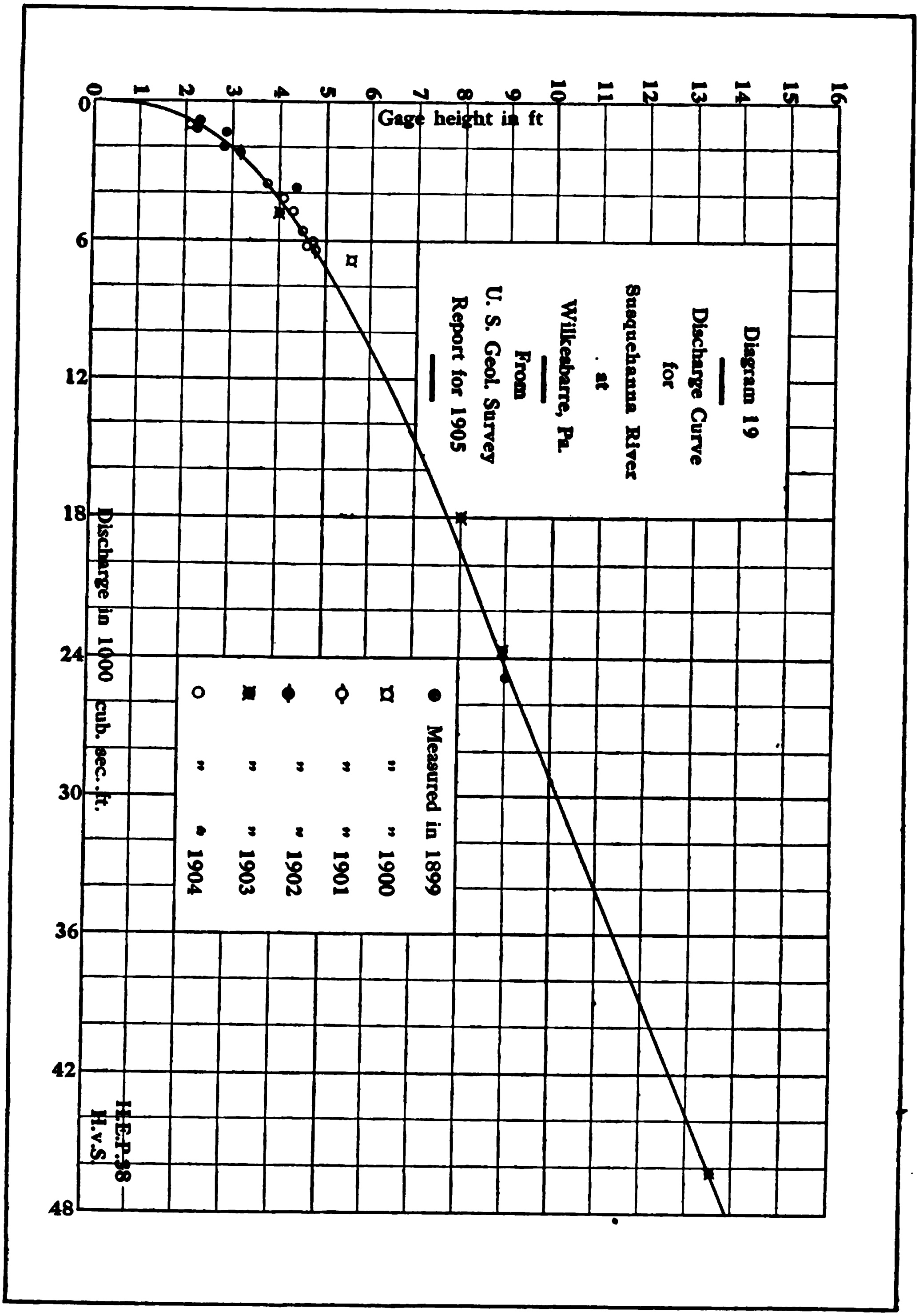


Diagram 19
Discharge Curve
for
Susquehanna River
at
Wilkesbarre, Pa.
From
U. S. Geol. Survey
Report for 1905
Gage height in ft
Discharge in 1000 cub. sec. ft.
Measured in 1899
" " 1900
" " 1901
" " 1902
" " 1903
" " 1904
H.E.P.38
H.v.S.
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16
0 6 12 18 24 30 36 42 48

ARTICLE 43.—When the stream is small, it may be practicable to ascertain the flow by *weir measurement*.

Fig. 11 shows the elevation and section of a weir, being a rectangular opening of horizontal base and vertical ends, the edges of the weir crest and ends being wedge shaped. The weir crest should be of such height that the downstream water surface is below it. The theory of measuring the volume passing over the weir is based upon the law of velocity of flow discovered by Torricelli, to wit: "the theoretic velocity of the flow of water is like that of a body falling freely in a vacuum through a height equal to the head," – $V = \sqrt{2\,g\,h}$, where V is velocity in feet per second, g is acceleration of gravity, 32.2 feet per second, and h is the head or height of water above the weir crest.

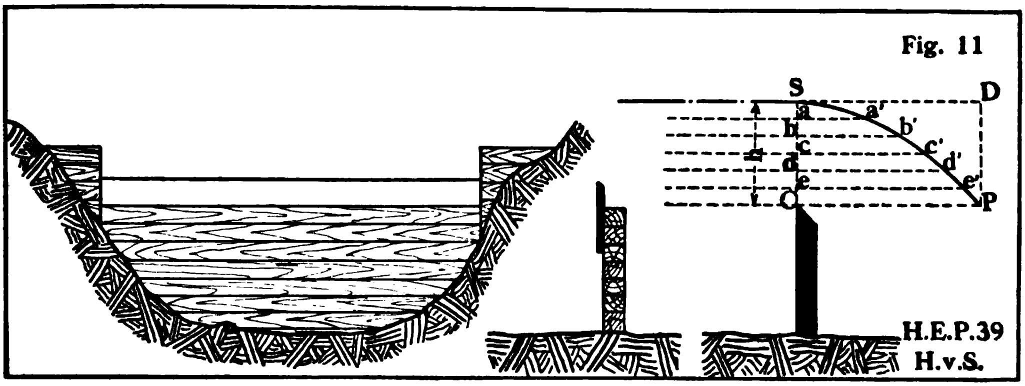


Fig. 11

In Fig. 11, C is the weir crest, S the surface of the water; each film of water passes with a velocity due to the head acting upon it,—*i.e.*, of 0.1, 0.2, 0.3, etc., feet down to the lowest film, the one near the crest, the velocity of which is that due to S C = h. Were the respective film velocities projected as ordinates a a′, b b′, c c′, d d′, etc., they would terminate in a parabolic line S a′ b′ c′ d′ e′ P, and the volume passing in a unit of time would be represented by the parabola segment S P C. In accordance with the geometric theorem, the area of this segment is two-thirds of the rectangle of like base and altitude, or $S\,P\,C = \frac{2}{3}\,h \times \sqrt{2\,g\,h}$, which therefore expresses the theoretic volume passing over the weir.

This value is based upon the free falling of the water in a vacuum, and the actual volume will therefore be reduced from this theoretic by reason of the friction of water against the weir crest and ends and against the air, all of which retarding influences are expressed by coefficients

which have been determined from results of many experiments. End contractions are expressed by J. B. Francis in a reduction of length "L" of weir = 0.1 h for each such contraction, while the other reductions from the theoretical volume are expressed by the same authority by a coefficient

$$M = 0.622.$$

More recent determinations by M. H. Bazin differ slightly from this. The weir formula is then

$$Q \text{ (discharge)} = M \tfrac{2}{3} h \sqrt{2gh} \times (L - 0.2\,h\,L).$$

$$\text{Solving, } Q = 0.622 \times \tfrac{2}{3}\, 8.02\, h \sqrt{h} \times (L - 0.2\,h\,L).$$

When L exceeds 5 h the correction for end contractions may be omitted, and for the purposes of stream measurements

$$Q = 3.33 \sqrt{h^3} \text{ per linear foot of weir,}$$

where "h" represents the height of water on the weir crest measured at a point upstream of the weir and above the initial depression due to the overfall.

Diagram 2 is constructed from this formula.

ARTICLE 44. *Flow deduced from Precipitation and Evaporation.*—When flow measurements are insufficient to yield a rating curve, especially when the low flow remains uncertain, the only method by which an approximation of it can be found is by the deduction of the run-off as the difference between the precipitation and evaporation. The general theory on which this method is based has been outlined in Art. 16, Part I. The detail operations of its practicable application are as follows:

Evaporation computations are based upon the determined monthly ratios due to the requirements of vegetation, the capacity and condition of ground storage, and the temperature. The year, for this purpose, is divided into two periods, the first being from December to May, when vegetation needs but little moisture and the evaporation is only that due to the action of the sun; during these six months evaporation is small and fluctuates only with precipitation or is very similar to evaporation from water surfaces. The quantity of evaporation during this period is approximately expressed by $E = 4.20 + 0.12\,R$, in which "E" represents

total evaporation and "R" total precipitation during this period. The second period is from June to November, when vegetation matures and requires a large amount of moisture; this is generally expressed by $E = 11.30 + 0.20\ R$.

The monthly distribution of these quantities is given by the following values:

For December	$e = 0.42 + 0.10\ r$
For January	$e = 0.27 + 0.10\ r$
For February	$e = 0.30 + 0.10\ r$
For March	$e = 0.48 + 0.10\ r$
For April	$e = 0.87 + 0.10\ r$
For May	$e = 1.87 + 0.20\ r$
For June	$e = 2.50 + 0.25\ r$
For July	$e = 3.00 + 0.30\ r$
For August	$e = 2.62 + 0.25\ r$
For September	$e = 1.63 + 0.20\ r$
For October	$e = 0.88 + 0.12\ r$
For November	$e = 0.66 + 0.10\ r$

"e" is monthly evaporation, "r" monthly precipitation.

These values were found substantially correct for the latitude of New Jersey, while for others a temperature correction is to be applied. Five per cent. for each degree of temperature as differing from that normal in the latitude above referred to appears to correct these values for stream systems in other latitudes, and it is sufficient to apply this correction as based upon the mean *annual* temperature of the drainage area in question, which correction is expressed by $0.05\ T - 1.48$, in which "T" is annual temperature; this may be termed the *temperature factor*, with which the values above given for monthly evaporation are to be multiplied.

ARTICLE 45.—*A typical case of flow determination* will now be taken up and argued month by month to its conclusion. The river taken is the Maitland in Ontario, emptying into Lake Huron at Goderich, which was examined by the author in 1905, the flow being measured for a sufficient period to prove substantial agreement between the two methods. The year taken is 1903. Monthly precipitation records were available for five well-distributed stations in the area, and, as the stream flows generally westerly and its drainage area is located in one precipitation belt, the monthly mean of all stations was adopted.

The mean annual temperature was found to be 46° F.

Temperature factor $0.05 \times 46 - 1.48 = 0.82$.

Month.	Precipitation.	Evaporation.
December '02	2.18	$(0.42 + 0.10\ r) \times 0.82 = 0.52$
January '03	1.36	$(0.27 + 0.10\ r) \times 0.82 = 0.33$
February '03	1.80	$(0.30 + 0.10\ r) \times 0.82 = 0.39$
March '03	1.19	$(0.48 + 0.10\ r) \times 0.82 = 0.49$
April '03	0.89	$(0.87 + 0.10\ r) \times 0.82 = 0.79$
May '03	2.74	$(1.87 + 0.20\ r) \times 0.82 = 1.98$
June '03	2.85	$(2.50 + 0.25\ r) \times 0.82 = 2.63$
July '03	2.68	$(3.00 + 0.30\ r) \times 0.82 = 3.12$
August '03	2.87	$(2.62 + 0.25\ r) \times 0.82 = 2.74$
September '03	3.54	$(1.63 + 0.20\ r) \times 0.82 = 1.92$
October '03	3.92	$(0.88 + 0.12\ r) \times 0.82 = 1.11$
November '03	0.96	$(0.66 + 0.10\ r) \times 0.82 = 0.62$
December '03	4.28	$(0.42 + 0.10\ r) \times 0.82 = 0.69$

Having found the monthly evaporation, it is evident that the excess of precipitation over evaporation represents the run-off; but an examination of these two reveals the fact that during July evaporation exceeds precipitation; this generally will be the case during several months and apparently there would be no run-off under such conditions. This, however, is not so, at least only rarely on western streams, which sometimes dry up entirely during seasons of drought; but there is water stored in the ground and in lakes and swamps, and whenever the evaporation is greater than precipitation, and in fact when the excess of precipitation becomes small, water stored in the ground feeds out into the stream.

It would be well, at this stage, for the reader to retrace his steps and review the discussion of drainage area, its topography, geology, flora, and culture, as the important influences of all of these conditions are about to become more apparent.

The supply from which a stream is fed during periods when precipitation only slightly exceeds evaporation, or when there is no such excess, which is the case on almost every system during the growing season from June to September, depends on the drainage area characteristics: if the ground contains no storage, there can be no supply; if the storage is large, the supply from it will be correspondingly plentiful, yielding frequently as much, and a greater flow than a normal rainfall would furnish. The rainfall is generally very small during two or three months of this growing season: a wet summer is an exception rather than the rule.

From a generalization of drainage area characteristics, as represented by topography and geology, the investigations in this field have

resulted in a classification of

(1) An area of bold relief and in highlands with no surface storage.

(2) An area of drift-covered rock with no surface storage.

(3) An area of deep drift and large surface storage.

Others could be added as being descriptive of conditions between these, but in practice the investigator will generally find that the area under examination is practically described by one of these three classes. It is evident that the ground storage capacity of these three will greatly differ, and, as they do, they will be capable of furnishing a comparative supply to the stream during the periods of small rainfall.

In the course of the search for a practical determination, fixed values of ground flow from areas of these different classes have been found and are represented by *ground-flow diagrams on Profile* 2. The side notations stand for monthly flow in inches from the ground storage, while those at the bottom represent the corresponding depletion of the ground storage, also in inches. Examining the projected curves, their more gradual flattening will be noted as the storage capacity of the area increases, while all finally assume almost the horizontal, indicating that storage is nearly depleted; it will also be seen that each of these begins to feed out with a flow of two inches, which means that, whenever the excess of precipitation over evaporation is less than two inches, ground storage begins to supply in accordance with the quantity of the remaining storage. The essence of all of this ground storage topic may be expressed by the following:

The ground storage conserves the excess precipitation and from it feeds to the stream during dry seasons in accordance with its capacity.

We may now go back to the finding of the flow on the Maitland River, and we note that during the very first month the excess of precipitation over evaporation is less than two inches, and, from what has been said before, we know that ground storage will add some supply and the storage itself will be correspondingly depleted. The ground-flow diagram arrangement shows what the depletion corresponding to a certain outflow from ground storage is, which may be expressed as d_1 = depletion or condition at the end of the month preceding the one under consideration, when d_2 for present month $= d_1 + e + f - r$, being the sum of existing depletion and present month's evaporation and flow less precipitation,

and for the average condition of the month

$$d = \frac{d_1 + d_2}{2}, \text{ or}$$

$$d = d_1 + \frac{e - r}{2} + \frac{f}{2}, \text{ or}$$

$$d = \frac{f}{2} + d_1 - \frac{r - e}{2}.$$

Applying this to December, 1902, where

$$r = 2.18, \text{ and } e = 0.52, \frac{r - e}{2} = 0.83,$$

there being no previous depletion, therefore $d_1 = 0$ and

$$d = \frac{f}{2} - 0.83.$$

The ground-flow diagram for Maitland River is that of a drainage area with drift-covered rock and no surface storage (No. 2), and, examining the curve, we find that the intersection of $d = 0.25$ and $f = 2.15$ fills the condition because $\frac{f}{2} = 1.07$ and $\frac{f}{2} - 0.83 = 0.24$; "f" is therefore 2.15 inches. The difference between the sum of evaporation and flow and of precipitation must come from ground storage and $d = e + f - r$, or in this case

$$0.52 + 2.15 - 2.18 = 0.49.$$

When the next month, January, 1903, is examined, $r = 1.36$, $e = 0.33$, and ground storage is depleted by 0.49 inch.

Then from

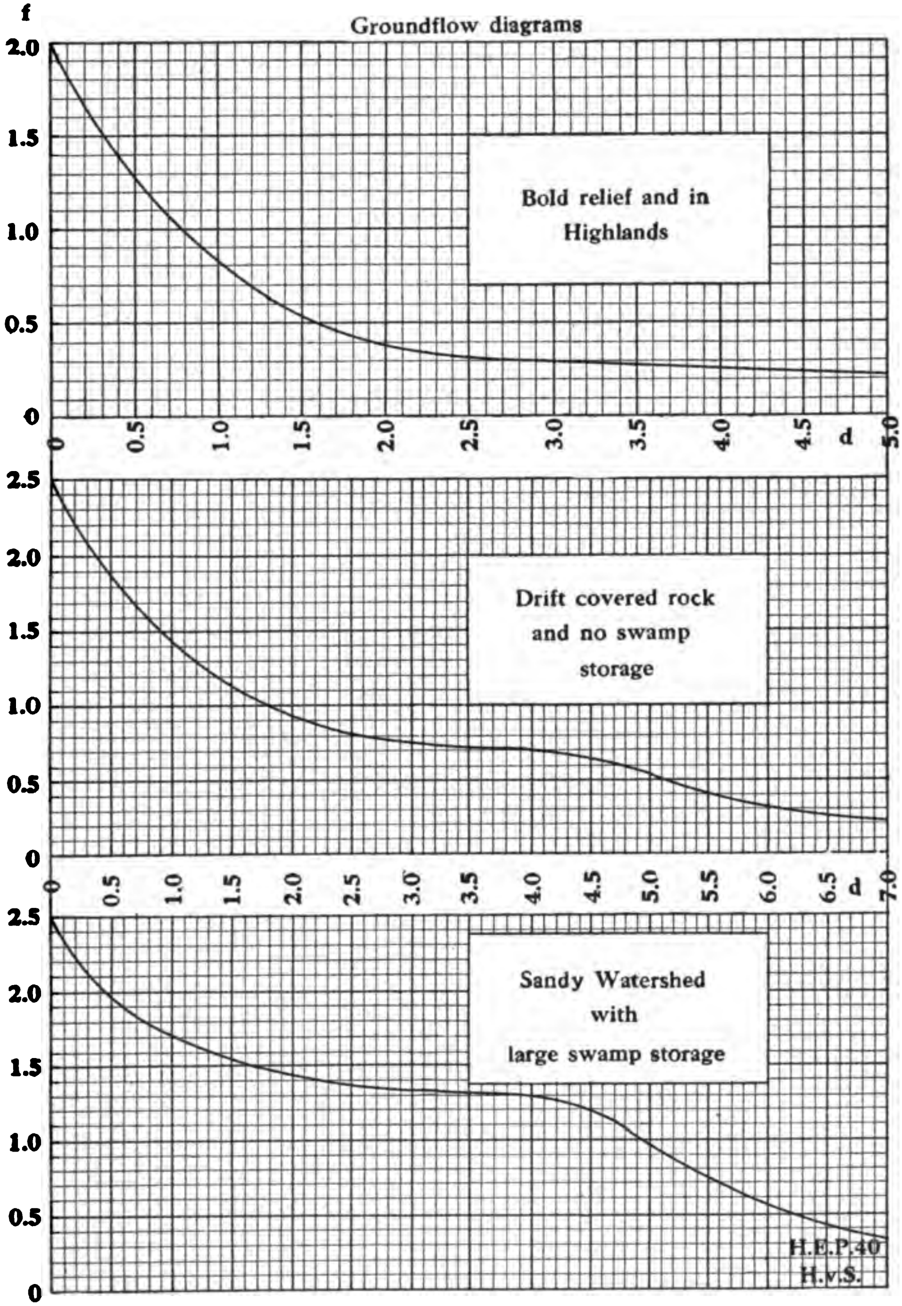
$$d = \frac{f}{2} + d_1 - \frac{r - e}{2}$$

$$= \frac{f}{2} + 0.49 - 0.52$$

$$= \frac{f}{2} - 0.03,$$

which is practically met by the intersection on the ground-flow diagram of 0.78 d and 1.62 f or $0.78 = 0.81 - 0.03$, and f therefore $= 1.62$, while depletion is again = to total evaporation and flow less total precipita-

Profile 2



From report of Geol. Survey of N. J. 1894 by C. C. Vermuele C. E.

tion = 0.85 + 3.76 − 3.54 = 1.07; or, in other words, the total supply represented by the total precipitation and the ground storage outflow (depletion) must always equal the total amount expended, that is total evaporation and total flow. As rainfall increases, ground storage again becomes gradually replenished, as appears at the end of the year. In this manner month by month is taken up and the run-off found in inches from the drainage area, which, for practical application, is later transposed into flow of cubic second feet.

The convenient arrangement of these deductions is as follows:

Column 1 gives the months, commencing with December of the previous year, because the accepted water year is from December to November.
Column 2 shows monthly precipitation .... r
Column 3 shows total precipitation .... R
Column 4 shows monthly evaporation .... e
Column 5 shows total evaporation .... E
Column 6 shows monthly run-off .... f
Column 7 shows total run-off .... F
Column 8 shows depletion of ground storage .... d

The computations are here given in detail in accordance with above arrangement, and Column 6 contains the resultant monthly run-off in inches per square mile of drainage area.

TABLE 3.—ORDINARY DRY YEAR MONTHLY RUN-OFF FROM MAITLAND, ONT., RIVER WATER-SHED. (All measurements in inches.)

1903 Month. 1	Precipitation: Monthly. 2	Precipitation: Total. 3	Evaporation: Monthly. 4	Evaporation: Total. 5	Run-off: Monthly. 6	Run-off: Total. 7	Ground Storage. 8	Remarks.
December '02	2.18	2.18	0.51	0.51	1.67	1.67	full	
January '03	1.36	3.54	0.33	0.84	1.03	2.70	full	T = 46°
February '03	1.80	5.34	0.39	1.23	1.41	4.11	full	
March '03	1.19	6.53	0.48	1.71	1.44	5.55	—0.73	Ground flow
April '03	0.89	7.42	0.77	2.48	0.76	6.31	1.37	taken as
May '03	2.74	10.16	1.93	4.41	0.60	6.91	1.16	from watershed
June '03	2.85	13.01	2.56	6.97	0.60	7.51	1.47	of bold relief
July '03	2.68	15.69	3.04	10.01	0.40	7.91	2.23	with *no*
August '03	2.87	18.56	2.67	12.68	0.33	8.24	2.36	swamp or lake
September '03	3.54	22.10	1.87	14.55	0.44	8.68	1.13	storage and
October '03	3.92	26.02	1.08	15.63	1.71	10.39	full	some drift
November '03	0.96	26.98	0.54	16.17	1.32	11.71	0.90	overlying
December '03	4.28	31.26	0.68	16.85	2.70	14.41	full	rock.

Article 46.—*Reservoir sites* should be looked for along the tributaries above the power site, and on lakes or swamps in the drainage area, and when found they should be surveyed to determine their available area and depth, location of reservoir dams, and cross-section at such.

Diagram 4 gives the continuous flow capacity, for various periods of time, in cubic-second feet, from an area of one hundred acres and one foot depth, from which the area corresponding to a required flow, or *vice versa,* can be taken.

Evaporation from reservoir surface, as per Table 4, Article 14, must not be overlooked, and some allowance should be made for water escaping from storage by seepage and by leakage through reservoir dam. The time required by the flow, from the storage reservoir to the power site, must also be determined.

Article 47.—The prevalence of *timber floating* down the stream, either from logging operations or trees on the banks which will be uprooted by the raising of the water above the dam, should be investigated; also the *ice* conditions during the winter periods, to what thickness it is likely to form and whether there is likelihood of its gorging in the river bends or above islands.

CHAPTER VII

DEVELOPMENT PROGRAMME

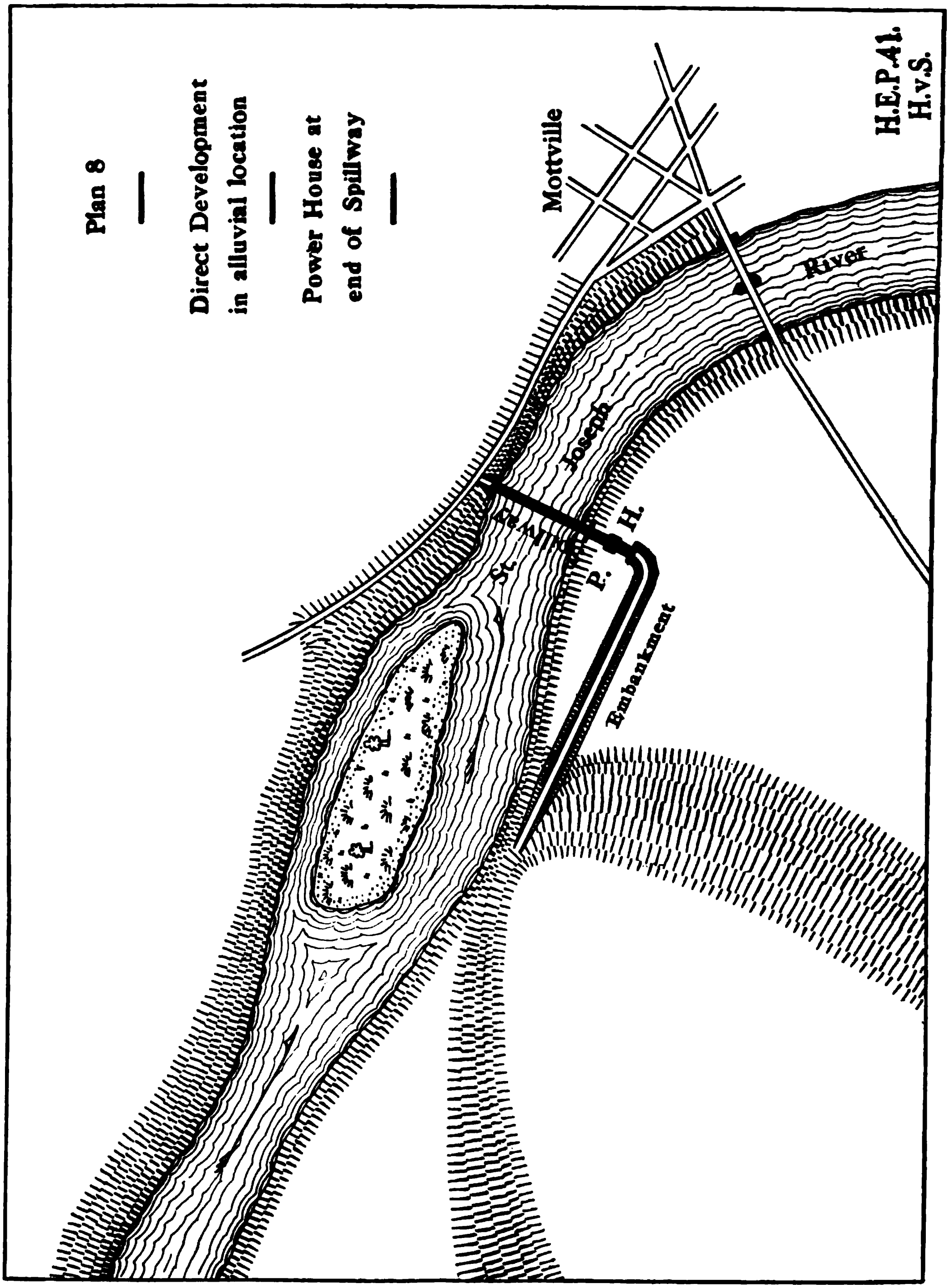
The data collected by the various operations described in Chapter VI. will furnish the information required to plan the best programme.

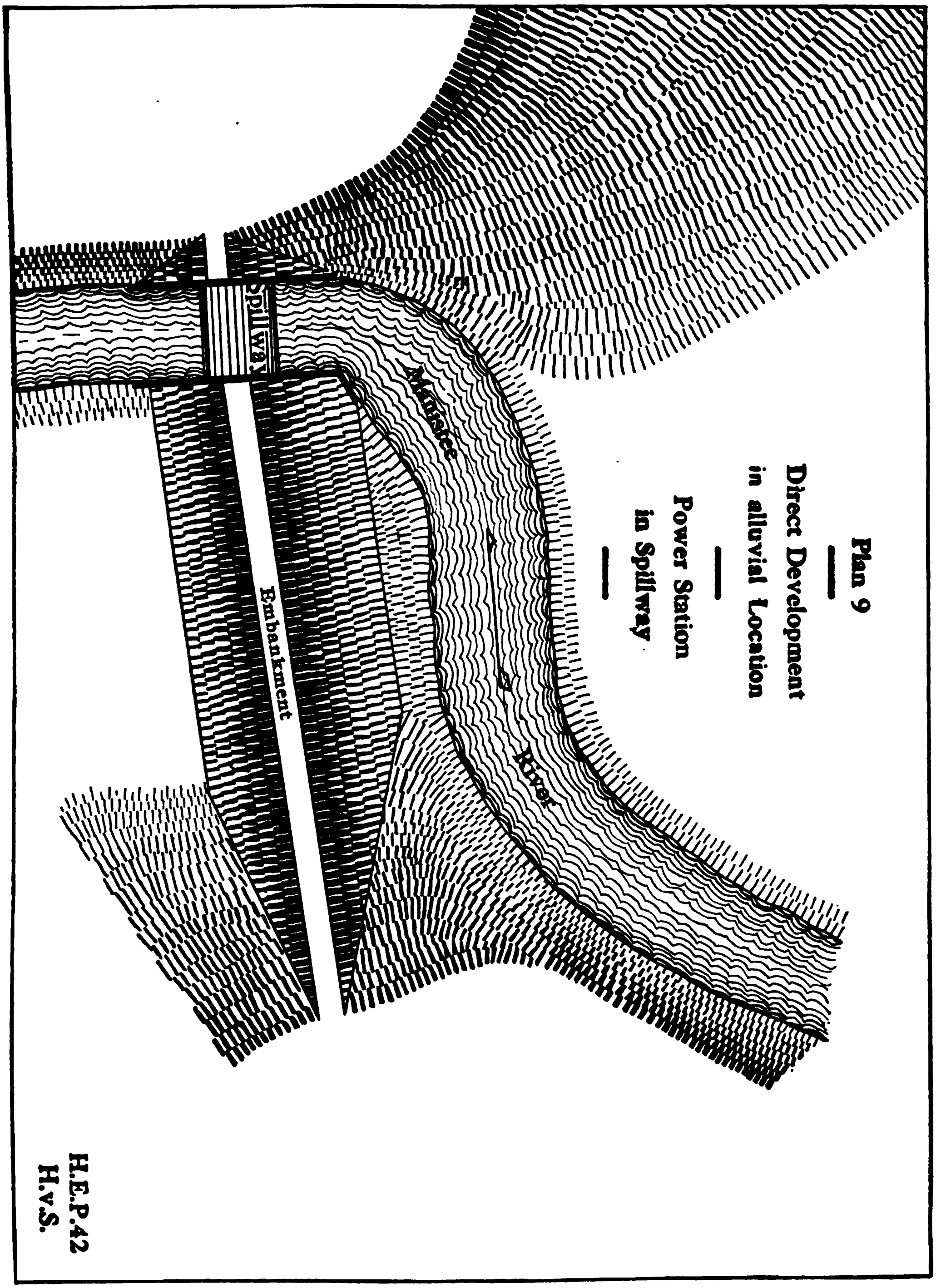
Article 48.—*The direct development* utilizes all the available fall at the dam, and the power station is located at its end or in the interior of the spillway. This plan is recommended because of the concentration of the entire plant at one point and the consequential saving in the operation cost, and because of its securing the highest obtainable hydraulic efficiency of the power components, fall and flow; by any other programme losses of both of these are incurred. Any diversion sacrifices a portion of the available fall by the slope in canals and flumes or the friction-head in pipe lines, while losses of flow are represented by leakage, evaporation, and ice conditions. When the water is passed at once from the upper pool through the turbines, no such losses occur. The conditions which determine this choice are the cost of the dam and embankments as compared with that of a lower dam and of diversion works; also the extent and cost of flowage for the upper pool and further the advantages secured by an extensive pond area; the flood flow conditions as affecting power house; the rise in the lower pool and the fluctuations in the working head. The rapid increase of cost with the height of the dam is shown on Diagrams 9 and 10, Art. 23, and of the foundations, if in alluvial location, and of abutments, on Diagram 11, Art. 23. When the location is in a narrow rock gorge, the entire width of the river will be required for the passage of the flood flow, and then it is not permissible to occupy any portion of it by the power house; to create a location for it in the rock bluff would be a costly undertaking. The solution for such a case may be found in arranging the interior of the spillway for the power station, as will be detailed further on, and in this way a spillway of the full river width becomes available and the direct development feasible. If the river is subject to frequent high stages, when the discharge over the spillway represents large volumes, it will correspondingly raise the level in the power-house pits and may impede the efficiencies of turbines; floating timber and ice also have a bearing upon this pro-

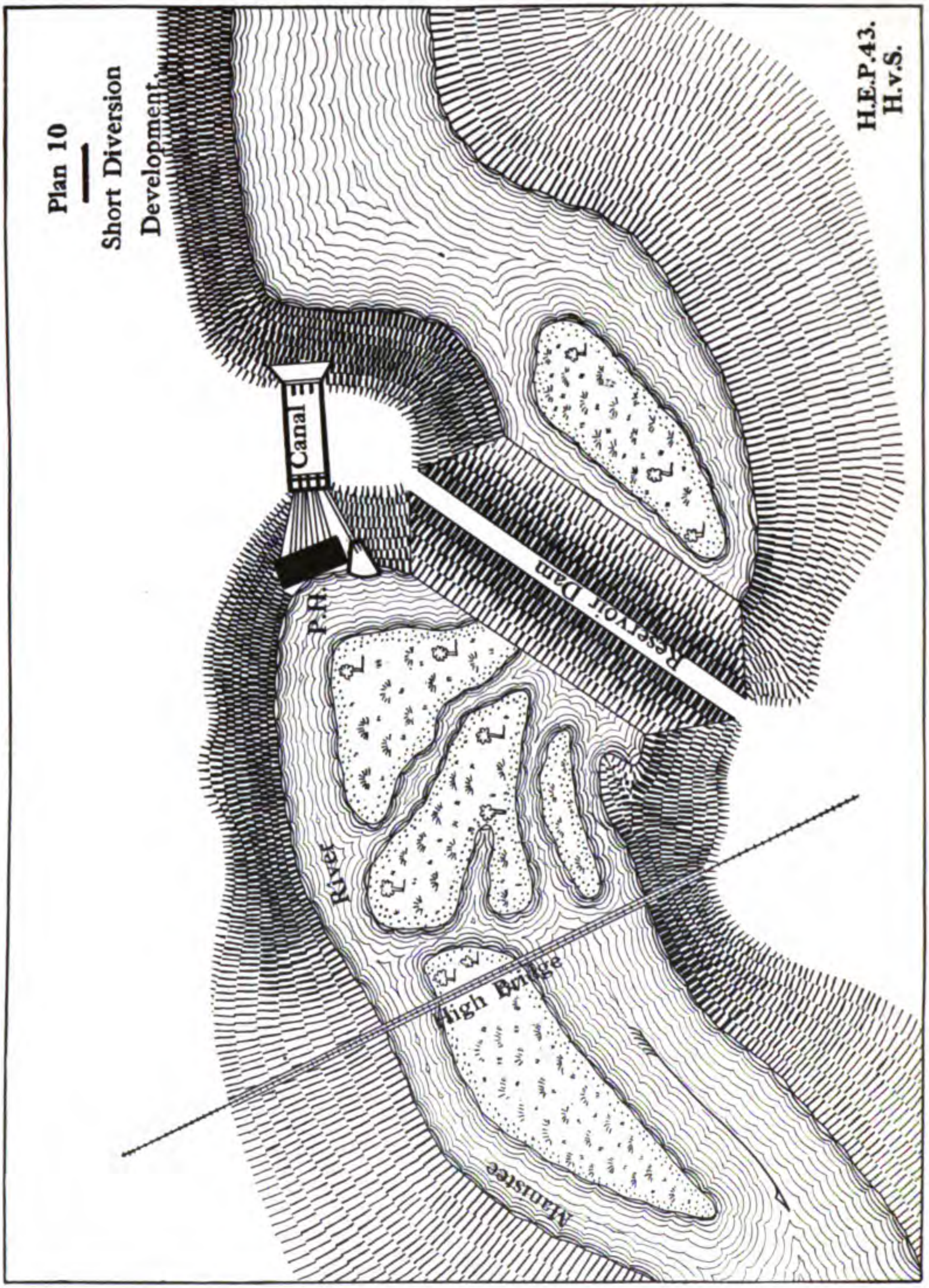
gramme, as it may necessitate costly safeguards to prevent injury to the power house or interference with the free entrance of the water to the turbine chambers. Thus, while the direct development plan realizes the highest percentage of flow and fall and represents the greatest simplicity of works and lowest operating charges, and therefore, as a rule, the most economical, the conditions may sometimes be such that its adoption is prohibited by the first cost or by considerations of safety and of continuity of operations.

ARTICLE 49.—*The short diversion programme* meets conditions where the dam location of the power house is not feasible because of contraction of the river channel or of the insufficient height of the spillway to accommodate the power equipment in its interior. The power house is then located as close below the dam as practicable, but at a safe distance from the spillway overfall. Water is conducted from the spillway pond in accordance with the volume to be utilized, in a canal, flume, or pipe. Since this programme is adopted only to escape the excessive cost or dangerous conditions, it presents more problems requiring careful solution than the former. No matter how short the diversion works, proper guards at point of intake are required, which, in combination with the spillway structure, cover a wide range of types. It may be advisable to locate the intake at some distance above the dam, in order to escape heavy rock cutting or ice gorges and to secure the most complete diversion of the low flow into it; to accomplish the latter object, on a wide river it may be necessary to provide a diverting dike or weir. When the rock bank continues precipitous for some distance above the dam, a partition wall may be required, or in some such cases it may be found to be most economical to arrange for diversion through a tunnel around the dam abutment. The intake entrance must be guarded by some kind of head gates, their character depending upon a number of controlling conditions which will be treated in detail later on. The diversion method will be shaped by the character and the formation of the river bank and the volume to be carried, detail considerations of which will be found under "Canals, Flumes, and Pipe Lines." The power house is placed at the most convenient point immediately below the dam; types will be described in the next chapter.

ARTICLE 50.—*The distant diversion* programme is applicable only when the concentration of the available fall at one point is not feasible or is too costly. The spillway or reservoir dam is located at the most


Plan 8
Direct Development
in alluvial location
Power House at
end of Spillway
Mottville
St. Joseph River
P. H.
Embankment
H.E.P.41.
H.v.S.


Plan 9
Direct Development
in alluvial Location
Power Station
in Spillway
Spillway
Embankment
River
H.E.P.42
H.v.S.

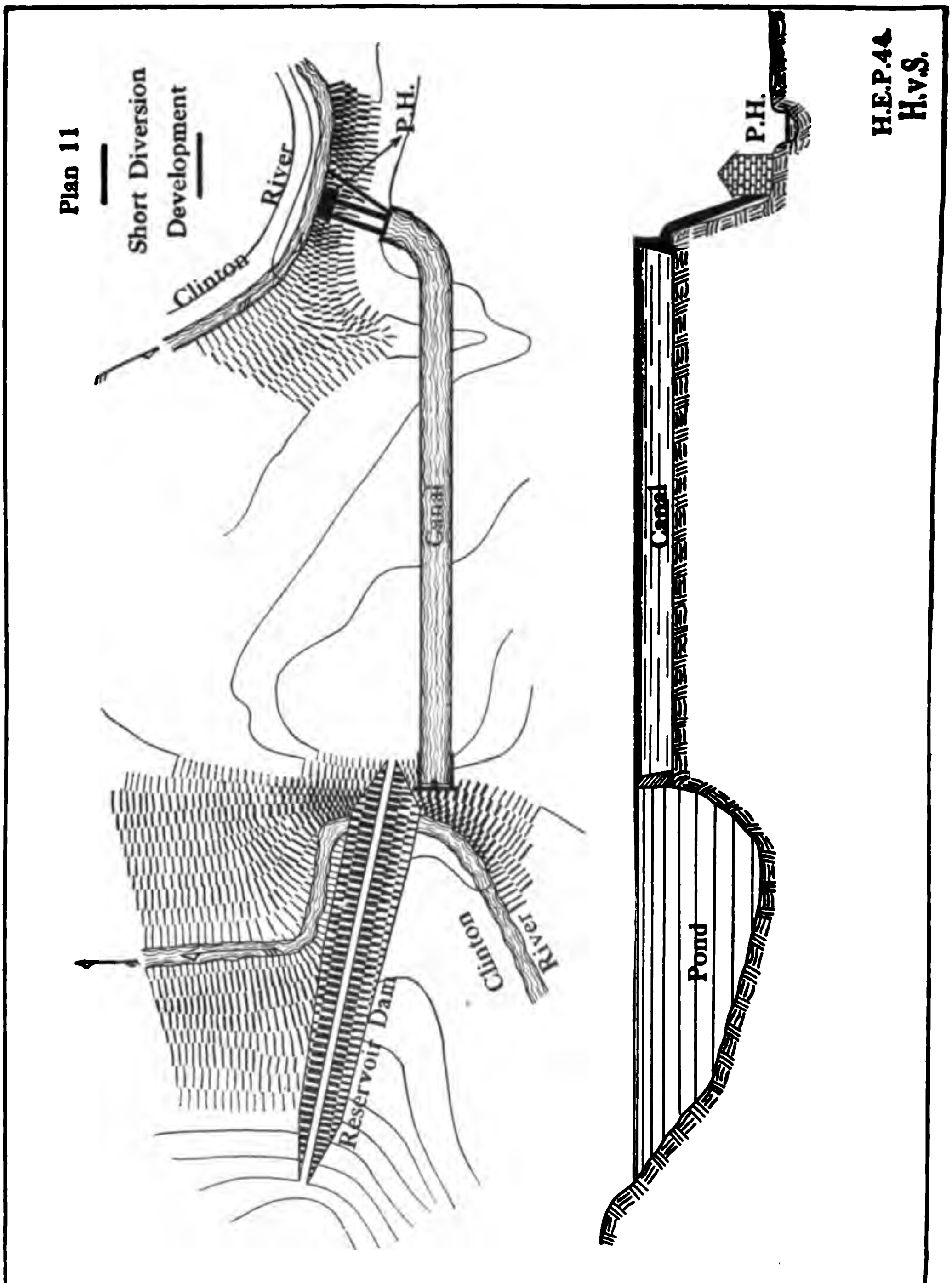

Plan 10
Short Diversion
Development.
Canal
P.H.
Reservoir Dam
River
High Bridge
Manistee
H.E.P.43.
H.v.S.

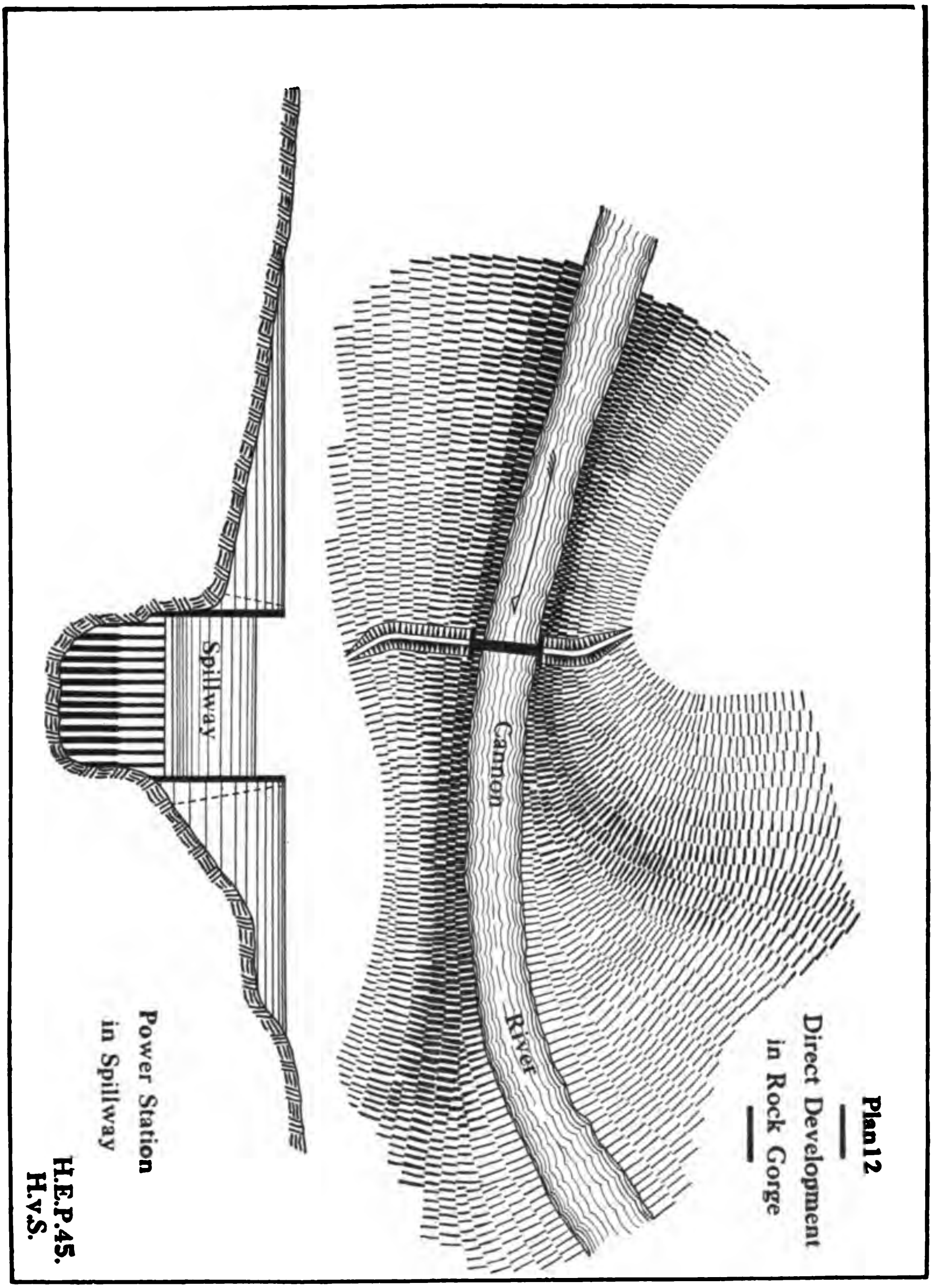
advantageous point, and the water is conducted from there to the lower level by a canal, flume, or pipe line, and the power station is at the terminal. The features of this class are very similar to those of the short diversion programme, the difference being only the distance of diversion.

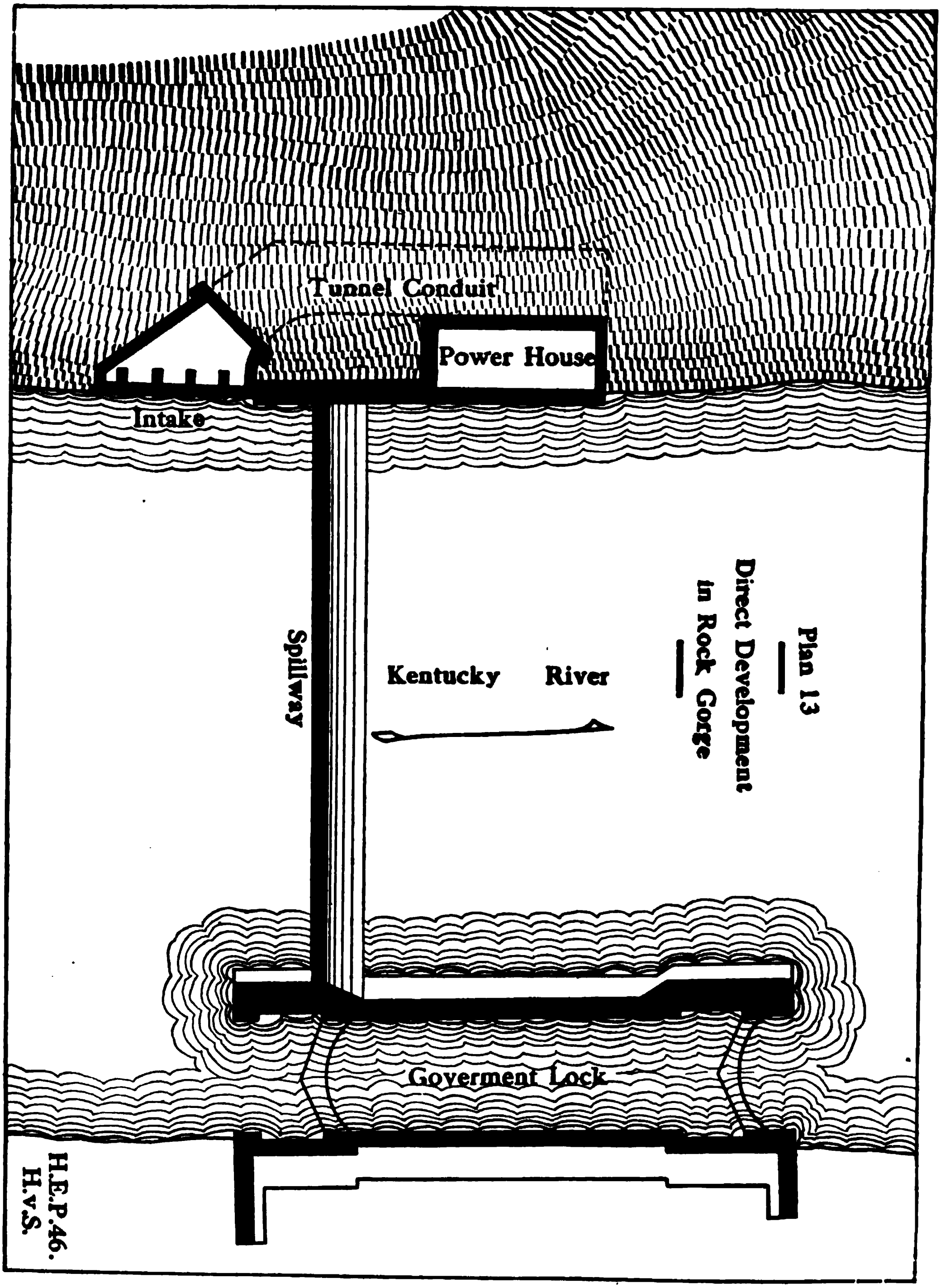
The choice of development programme is, as a rule, not a difficult problem; as the existing conditions in most cases readily point to one or the other, only occasionally may some doubt exist as between the first two.

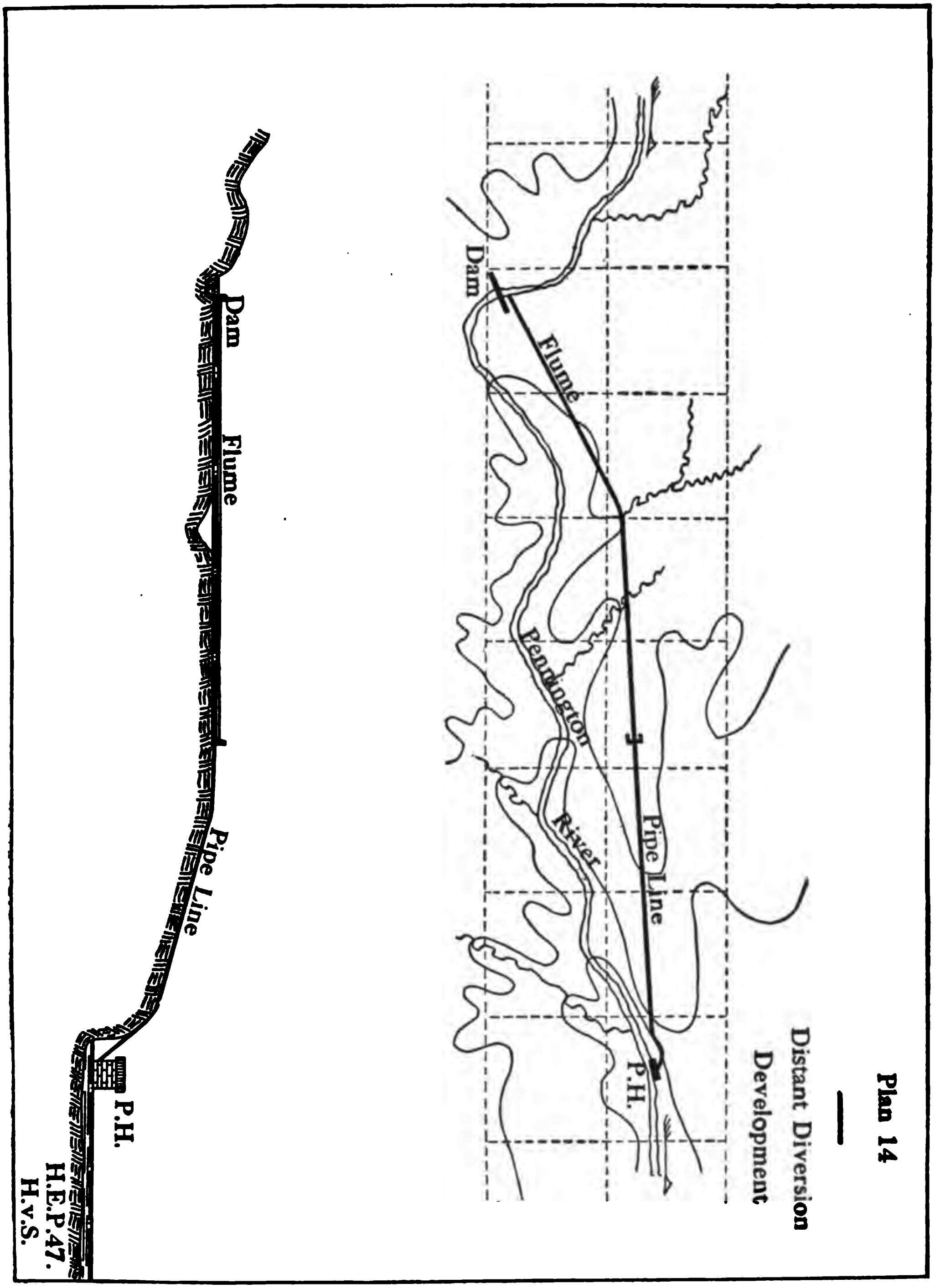
When the development is on the lower reach of a river without falls or rapids, aiming to concentrate so much of its natural fall as may be feasible with the available height of its banks, and when the formation is generally alluvial, the direct programme is the solution, the power house being placed at the end of the spillway, as is shown on *Plan* 8, of the *Mottville, Mich.*, development; or if this is 30 feet and higher, in the spillway's interior, as illustrated on *Plan* 9, of the *Manistee River, Mich.*, plant. On such locations high flood conditions may advise the adoption of the second programme, or the peculiar formation shown in *Plan* 10, *High Bridge, Mich.*, where a promontory juts out into the river, presents a favorable condition for the short diversion development. The third would be available only in case sufficient more fall could be secured by it, for instance when the river makes a long detour, doubling back to within a short distance abreast of the dam site, the diversion location being across the peninsula formed by the river's oxbow course. The fall of rivers of such characteristics rarely exceeds two feet per mile and the length of its course around the détour must be five miles or more to warrant such a programme; in fact, its advisability must be weighed by a comparison of the earning capacity of the fall thus gained and the investment represented by the cost of the diversion works plus their maintenance and operation. This case is illustrated on *Plan* 11, of the *Clinton River, Mich.*, Renshaw site development, where the stream departs easterly for a distance of five miles and returning approaches the dam site within 1200 feet, gaining 12 feet fall.

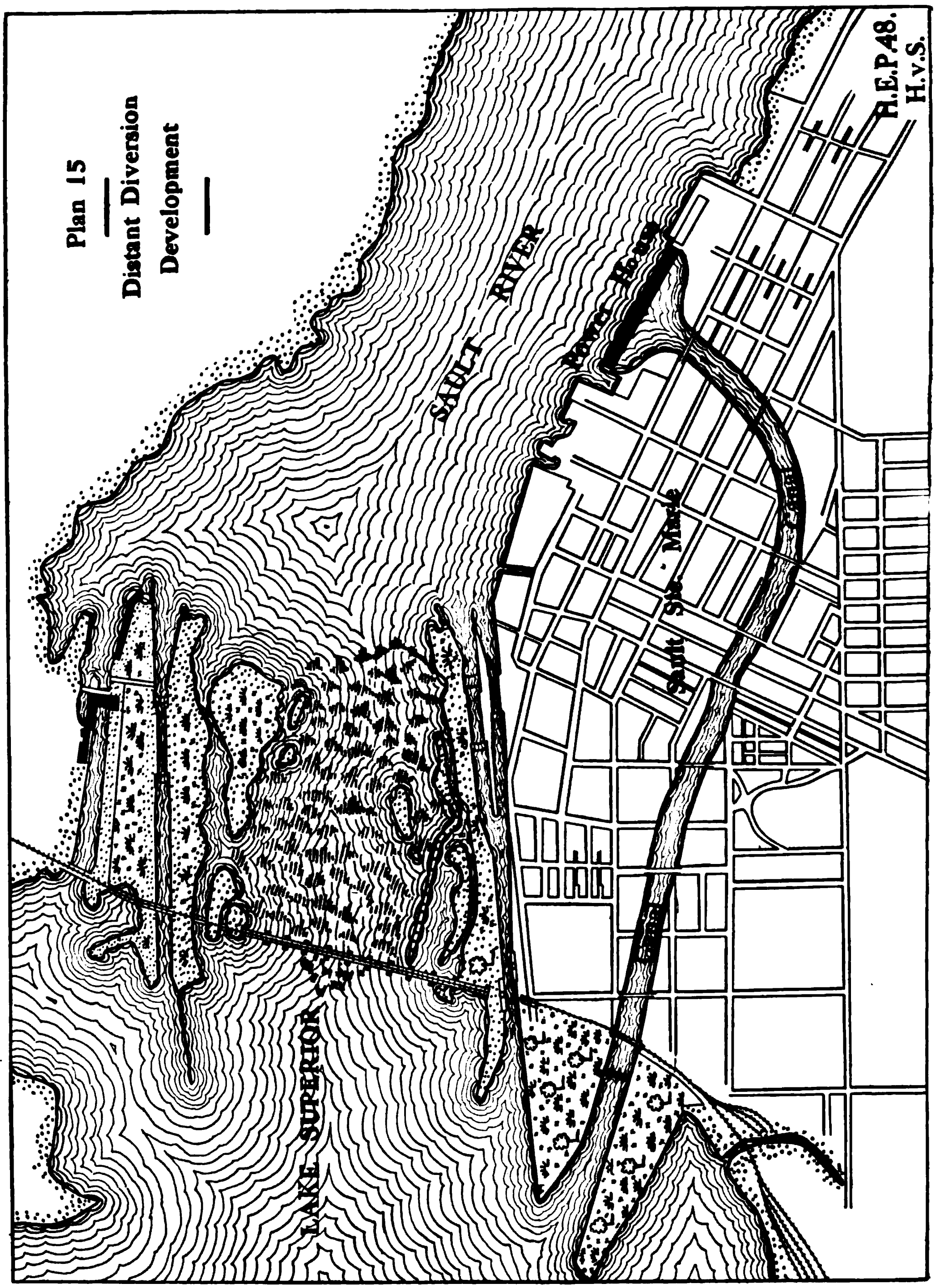
In rivers with rock beds and palisade banks the first programme is admissible only when the spillway's interior can be utilized for the power station, as the entire width of the river channel must remain available, unobstructed, for the passage of the flood flow, and the creating of a power-house site at the spillway end involves the removal of rock and would be very costly. Such a development is shown on *Plan* 12, of the

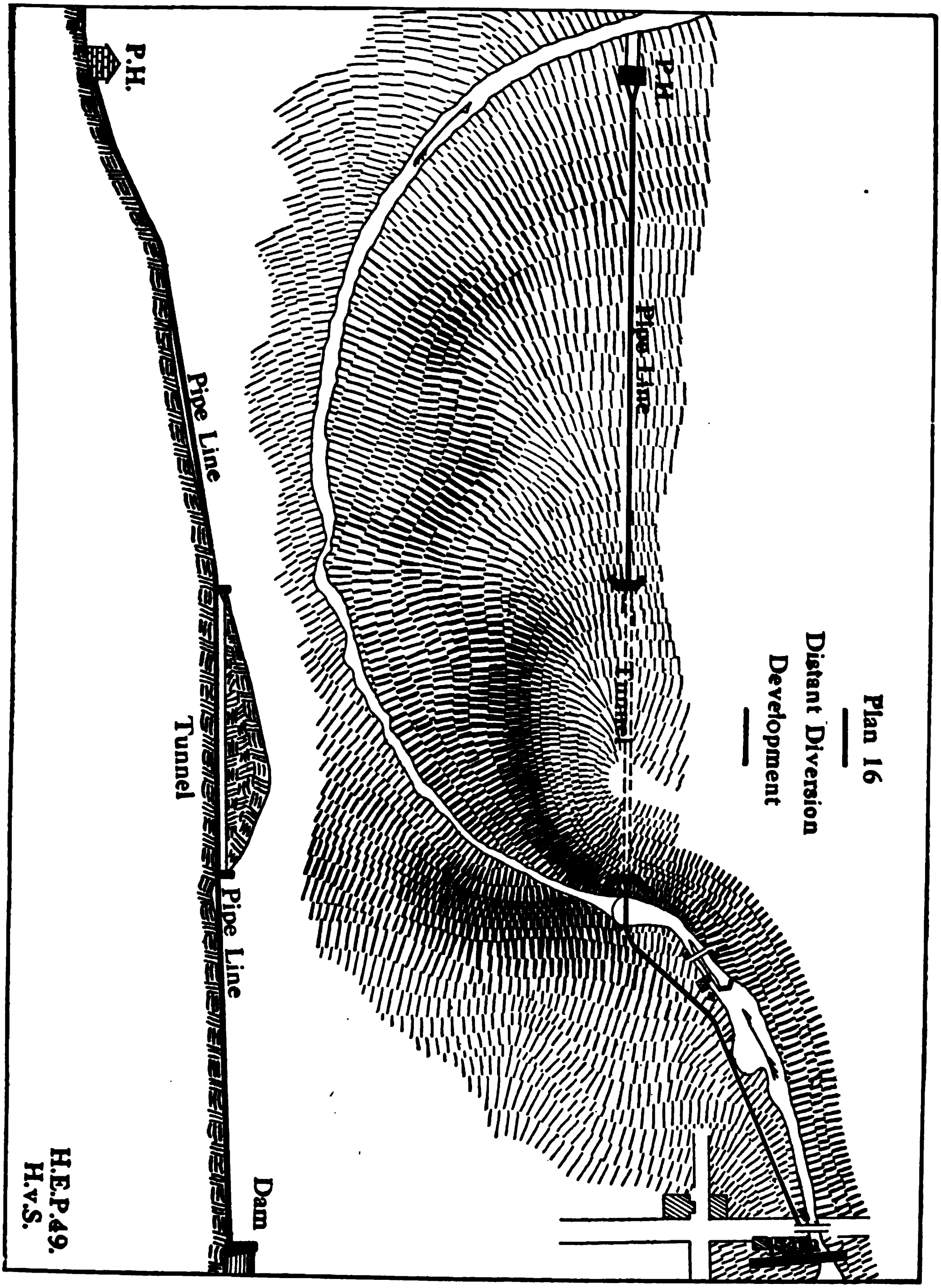

Plan 11
Short Diversion
Development
Clinton
River
P.H.
Canal
Reservoir Dam
Clinton
River
Canal
P.H.
Pond
H.E.P.44.
H.v.S.

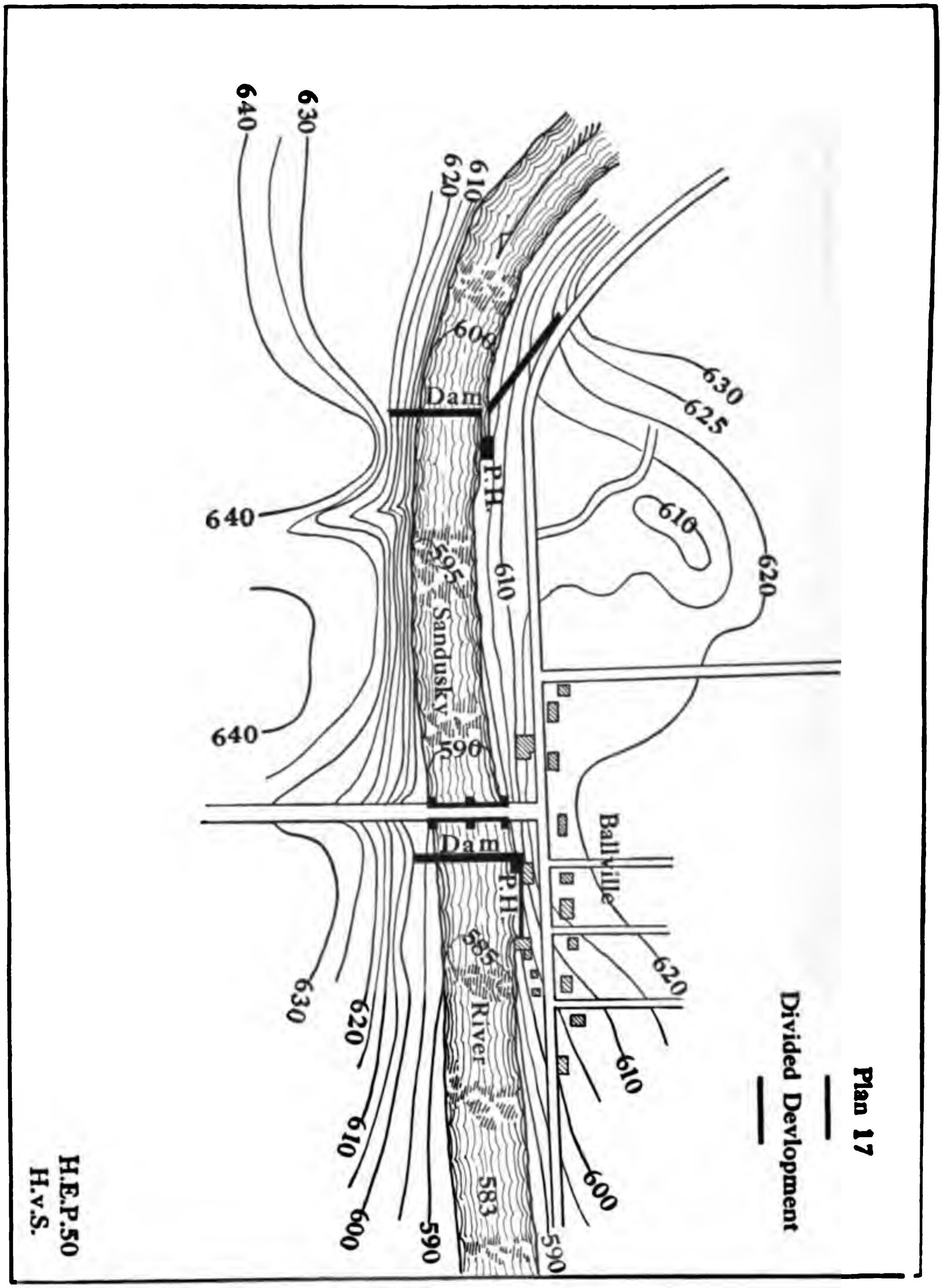

Spillway
Cannon
River
Plan 12
Direct Development
in Rock Gorge
Power Station
in Spillway
H.E.P.45.
H.v.S.


Tunnel Conduit
Power House
Intake
Spillway
Kentucky River
Plan 13
Direct Development
in Rock Gorge
Goverment Lock
H.E.P.46.
H.v.S.


Plan 14
Distant Diversion
Development
Dam
Flume
Pennington
River
Pipe Line
P.H.
Dam
Flume
Pipe Line
P.H.
H.E.P.47.
H.v.S.


Plan 15
Distant Diversion
Development
SAULT RIVER
LAKE SUPERIOR
Sault Ste. Marie
H.E.P.48.
H.v.S.


Plan 16
Distant Diversion
Development
P.H.
Pipe Line
Tunnel
P.H.
Pipe Line
Tunnel
Pipe Line
Dam
H.E.P.49.
H.v.S.


Plan 17
Divided Devlopment
H.E.P.50
H.v.S.
Dam
P.H.
Dam
P.H.
Sandusky
River
Ballville
640
630
620
610
600
595
590
585
583
625

Canon Falls, Minn., plant, the river forming practically a rock gorge, the spillway 40 feet high, and the power station inside of it; this can be carried out only when the spillway is 25 feet or higher, and the only alternative is as shown in Plan 13, at *Little Hickman*, Green River, Ky., where the power house is 200 feet below the dam, diversion in this case being by tunnel.

When falls or rapids continue over a considerable distance, the third programme only is available,—that is, if a fall exceeding the feasible accumulation at one point is to be utilized. *Plan* 24, of the *Pennington, Ind. Ter.*, illustrates this, the fall in 3½ miles being 135 feet; diversion is by flume and pipe line; also *Plan* 15, of the development at *Sault Ste. Marie, Mich.* The development of most of the high falls is of this type, as appears on *Plan* 16, of the *Eugenia Falls, Ont.*, where a vertical drop of 78 feet is followed by continuous rapids in the stream flowing for one mile around a rock bluff; the spillway here is placed above the fall, and ¾ mile diversion by tunnel and pipe line terminates at a point 400 feet below the spillway crest. Occasionally successive falls can only be developed by separate treatment,—that is, topography or right-of-way limitations prohibit any diversion programme. This is the case on the Sandusky River near *Fremont, O., Plan* 17, where a fall of 40 feet occurring in half a mile can be developed only by two separate dams within a quarter of a mile of each other. Only the best market conditions will warrant such a treatment when the operating cost of the two stations creates a heavy charge against the enterprise.

Article 51.—*Development scope—i.e.*, what output capacity is the development to be based upon—should be determined before the plant is designed. This question is be decided from the considerations of the available market for the product and of the fall and flow which can be utilized. The market topic has been discussed in Part I. In the direct development it is generally advisable to utilize all the available fall; with distant diversion the cost of the latter corresponding to the fall gained is the criterion. If only a part of the available power is to be developed, the decision whether to use partial fall or flow rests with the cost of works adapted to one or other purpose; this, however, is rarely the case; on the contrary, it will be almost uniformly desirable to develop the largest possible capacity, as the market for the product is sooner or later created, and the greater the output at a given site the more economical will be the unit development and the operating cost. An estimate

of the largest remunerative development is among the first questions met while the project is being exploited, and this must be based upon reliable data and should be as complete as practicable.

Logically, therefore, this determination of the scope of the development, as far as it influences the extent and character of the works and equipment, and consequently the probable total investment, belongs to the topics discussed in Part I., "Analyzing a Hydro-electric Project," but it has been necessarily given a place here, in Part II., because the various designs of works and their cost enter so largely into it.

This conclusion of the largest resourceful development implies that the available flow and fall are correctly determined. These represent the power source. The reliable procedure is to find the lowest annual precipitation during a cycle of fifteen years, the dry year, which is a matter of Government records, and to establish the daily flow during this year. When gaugings have not been made it becomes necessary to take those of some other water course in the same drainage basin and apportion the run-off to the ratio of the respective catchment areas.

For example of a river draining 1000 square miles at the point of the prospective development the following may be cited as a summary of the daily flow:

For Days	Flow in c. s. f.	Run-off per sq. m.	Total flow in c. s. f.
10	75	0.075	750
25	100	0.1	2,500
20	150	0.15	3,000
30	200	0.2	6,000
45	300	0.3	13,500
50	400	0.4	20,000
35	500	0.5	17,500
60	600	0.6	36,000
65	700	0.7	45,500
25	800	0.8	20,000
365	Aggregate		164,750
	Daily mean		451.37
	Daily run-off		0.45

In reality this tabulation will probably have one hundred instead of the above ten items. This is the flow record of the dry year and must be taken as the basis for available flow determination. It would be disastrous to the enterprise to take a year of greater flow, as this dry year has occurred and will occur again, and possibly two such in succession, of which there is ample evidence.

The low daily flow is 75 c.s.f., the high daily flow is 800 c.s.f., and the mean daily flow is 451 c.s.f. These represent the normal low fluctuations of eastern and northern streams; in the west and south they will be greatly exceeded.

It is evident that a flow basis greater than 75 c.s.f. could be maintained only by providing some supply when the flow is deficient; this may be accomplished by storing a portion of the higher flow and drawing from it during low periods. How far this can be done depends primarily upon the availability of suitable reservoir sites and the investment value of the required lands and works for the purpose of furnishing a flow supply for power production. The first must be ascertained through intelligent reconnaissance and ultimate surveys of located reservoir sites to establish their storage capacities and the required structure to impound the water The second may be found from an analysis of the daily flow summary by ascertaining the required supply to raise the low flow during the period when it prevails to the next higher. From daily flow tabulation it appears that for ten days the flow is 75 c.s.f.; to raise this to the next higher of 100 c.s.f. requires the addition of 25 c.s.f. for ten days. This supply is converted into storage units, the acre foot. One acre contains 43,560 square feet and one day, 86,400 seconds; an acre of water one foot deep therefore practically represents the aggregate volume of a 24-hour flow of one-half c.s.f. From Diagram 4 on page 30 the continuous flow in cubic second feet for any period of time and storage capacity, as well as the required supply to accumulate any storage volumes, may be readily taken. Evaporation, normal in the latitude of the Middle States, is allowed for, as well as ordinary seepage.

For the case in hand, a supply of 25 c.s.f. for 10 days, the storage capacity must be 500 acre feet. This is required to maintain the higher flow of 100 c.s.f. during ten days. This flow of 100 c.s.f. is now the lowest in the year; to raise it to the next higher, 150 c.s.f., requires a supply of 50 c.s.f. for 35 days, or a storage capacity of 3500 acre feet in addition to the last-found 500 acre feet, being a total of 4000 acre feet to maintain a flow of 150 c.s.f. as the lowest available. In this manner the year's flow analysis now becomes:

For Days	Natural flow	Storage Flow Required to Raise to Higher	Total Storage Required
10	75		
25	100	500	500
20	150	3500	4000
30	200	5500	9500
45	300	17000	26500
50	400	26000	52500

As the daily mean for the year is less than the next higher the investigation need not be carried further because any greater storage volume could not be relied upon during a cycle of two successive years. With an available storage of some 52,000 acre feet the entire dry year's flow could be utilized for power development; but the question is to be settled whether it will pay, whether the required investment for reservoir lands and works can be safely saddled upon the enterprise. One or more reservoirs of an aggregate of 2600 acres and an average storage depth of 20 feet would meet the requirement. Or a flow of 300 c.s.f. could be maintained with storage capacity of 26,000 acre feet; reservoirs of 1300 acres area with average storage depth of 20 feet would answer. These are the fundamental data for the available flow investigation, but any further step toward a final determination must consider practically every feature of the proposed development, because the scope of storage can be fixed only by knowing its investment value, by balancing the power output increase thus obtained against the cost of securing it. For this the head must be determined and the development program, type of works and equipment, cost of development and transmission, operating cost, fixed charges and finally the value of the product in the selling market. In other words the investment value of the development must be found for each stage of the various daily flow volumes, and as this is raised the cost of the required reservoir lands and works, of the increased diversion conduits, generating and transmission equipment are added. Thus the final cost of the product for the various flow bases is found, and the most resourceful will clearly be shown.

The following analysis of the potential flow which may be obtained from storage supplement will be helpful in arriving at a general estimation of the probable greatest continuous hydro-output.

POTENTIAL FLOW FROM ONE INCH PRECIPITATION PER SQUARE MILE DRAINAGE AREA PLUS PER CENT. OF CONSERVED FLOW

One inch rainfall per square mile accumulates......	2.323200 cu. ft.;
one year contains..............................	31.536000 seconds;
theoretical run-off represented by one inch rainfall is...................2.323200 ÷ 31.536000 =	0.07366 c.s.f.
Approximately two-thirds of this evaporates, or...	0.04911 c.s.f.,
while the remaining third represents the annual run-off, or..............................	0.02455 c.s.f.,

and one-third of this annual may be taken as the low run-off, or.	0.008 c.s.f.
100 per cent. conservation implies the storing of the total annual run-off in excess of the low flow which is always used for development, or from one inch rainfall on one square mile it means the storing of annual flow of. . 0.02455–0.008=	0.016 c.s.f.,
and the potential flow would be this, plus the low, or	0.02455 c.s.f.
90 per cent. conservation would be the storing of an annual run-off of. . . . . 0.016×0.90=	0.0144 c.s.f.,
and the potential flow, this plus the low flow, or.	0.0224 c.s.f.,

and so forth, which is the basis of the following table giving the potential flow secured by the partial conservation expressed in per cent. of the possible total.

To store an annual run-off of 0.016 c.s.f. requires

$$\frac{31.536000 \times 0.016}{43560} = \text{about } 11.6 \text{ acre-feet,}$$

and the storage capacities called for by other than 100 per cent. conservation is in ratio of the per cent.

Conservation, per cent.	Obtainable Potential Flow, c. s. f.	Required Storage Capacity, a. f.
100	0.02455	11.60
90	0.0224	10.44
80	0.0208	9.28
70	0.0192	8.12
60	0.0176	6.96
50	0.0160	5.80
40	0.0144	4.64
30	0.0128	3.48
20	0.0112	2.32
10	0.0098	1.16

Example.—For a drainage area of 1000 square miles and a dry year precipitation of 30 inches:

The total annual run-off is. . . . . 1000×30×0.02455 = 736 c.s.f.,
the low run-off. . . . . 1000×30×0.008 = 240 c.s.f.
For 20 per cent. conservation the
necessary storage capacity is. . 2.32×30000. . = 69,600 acre feet,

and if an average of 20 ft. storage depth can be had it requires reservoirs of.................3480 acres area.

This 20 per cent. conservation program would maintain a continuous lowest flow of.......$1000 \times 30 \times 0.0112 = 336$ c.s.f.,

or 94 c.s.f. greater than the naturally lowest flow.

Will the output increase due to this augmented flow warrant the cost of such a conservation program? If the effective head is to be 50 feet this added output will be about 350 e.h.p., representing an annual product of about 1,085,000 kilowatt-hours. If this can earn a surplus of half a cent per kilowatt-hour the net revenue from it is $5425.00, representing 7 per cent. (interest and depreciation) on an investment of $77,500.00. Will this cover the cost of the required reservoir site of 3480 acres and the needed reservoir dam, sluice, etc., and leave some margin? The solutions of such problems determine the scope of storage supplement.

If reservoir sites are not available the supplement of the low flow must be from auxiliary power supply. A second analysis of the daily flow summary is then made substituting power for storage for which the available head must be known. For example, in this case the effective head is fifty feet; the mechanical power yield of water power is

$$\frac{\text{cu. ft. of water} \times \text{weight} \times \text{head}}{550} \times 0.75,$$

and for one cubic foot it is

$$\frac{62.5 \times 50}{550} \times 0.75 = 4.26 \text{ h.p.}$$

The power supplement analysis then becomes thus:

For Days.	Natural Flow	Mechanical h. p. Represented by Higher	Total Auxiliary Power
10..................	75		
25..................	100	107	
20..................	150	213	320
30..................	200	213	533
45..................	300	425	958
50..................	400	425	1383

From this appears that it requires an auxiliary power plant of some 1400 h.p. to supplement the lower than 400 c.s.f. flow output in order to

maintain this constantly. This auxiliary plant would operate 135 days, the hydro source yielding a like or greater development during the remainder of the year. Here again, as in deciding upon storage scope, the investment value of the various auxiliary plants must be determined for the different stages into which all the details of the cost of the hydro-plant for full output, of auxiliary plant, fuel consumed, operating cost, maintenance and depreciation enter.

In some cases the development supplemented by auxiliary power may be carried beyond the mean daily flow, but these will be exceptional. Finally a combination of storage and auxiliary power supplements may sometimes prove resourceful; if, for instance, storage can be obtained economically which represents a capacity of ten thousand acre-feet, say a 500 acre reservior with an average storage depth of 20 feet. The highest flow which can be maintained from storage is 200 c.s.f. but by the addition of some auxiliary power supplement the continuous output may be further raised to a resourceful development, and for this program the following becomes the third analysis with the lowest storage supplied flow:

For Days.	Natural Flow	Mech. h. p. Represented by Higher	Total Auxiliary Power Supplement
85....................	200		
45....................	300	425	425
50....................	400	425	850
35....................	500	425	1275

From this it appears that the combination of a 1300 h.p. auxiliary plant and storage capacity of 9500 acre-feet will secure a continuous output of 2125 h.p., or that due to a flow of 500 c.s.f.

As has been said all of these programs may be analyzed as to their ultimate resourceful value, as appears from the following examples adapted to this case:

	1	2	3	4	5	6	7
	Output, m. h. p.	Flow Basis, c. s. f.	Supplement. Storage	Aux. Power	Cost	Charges	Current Cost
1.........	850	200	9500	....	120	27	0.6
2.........	1300	300	26500	....	147	36	0.4
3.........	1700	400	52500	....	188	46	0.5
4.........	850	natural	.....	550	148	33	0.73
5.........	1300	natural	.....	1000	170	47	0.51
6.........	1700	natural	.....	1400	198	61	0.68
7.........	1300	200	9500	450	165	42	0.45
8.........	1700	200	9500	900	192	60	0.66

Column 1 is the continuous output in mechanical horse power.
Column 2 flow utilized in c.s.f.
Column 3 flow supplement from storage of acre-feet capacity.
Column 4 output supplement from auxiliary plant in mechanical h.p.
Column 5 development and delivery cost in 1000 dollars.
Column 6 all fixed and operating charges in 1000 dollars.
Column 7 generating and delivery cost of current in cents per kilowatt-hour of full product.
Items 1–3 represent storage supplement only;
Items 4–6 represent auxiliary plant supplement only;
Items 7–8 represent the combination of both.

From this it appears that a development of 1300 h.p. with all storage supplement is the most resourceful, the cost of the output being the most economical (four tenths of a cent per kilowatt-hour); that the same output with storage and auxiliary power supplement is the next; and a like development with all auxiliary supplement the third. The ultimate determination now rests solely in the availability of reservoir sites; when this is established the most resourceful development is clearly determinable.

It is no doubt a fact that water-power developments are rarely thus analyzed when the project is promoted or even financed, that it is the general custom to fix upon some (ordinarily unwarranted high) hydro output, and that the question of supplementing the low output is left to the future when, it is said, the market demand for the product develops. That this is a most unwise procedure will now be clear to any attentive reader, and that it will rarely result in a final development which represents the most resourceful utilization of the power opportunity. The future addition of supplemental flow from storage or power from auxiliary plant will in almost every case call for changes and additions to works and equipment which will be more expensive and render the plant less efficient then if the original development program is planned for the most resourceful final conditions, even though the storage or auxiliary supplements are not actually incorporated until some later day.

This is the outline of the needful initial precautions in exploiting a water power development, and if they are followed by competent designing and construction the desired results can be relied upon. The best of plans, however, miscarry if, by shortsighted economy, their execution is delegated to inexperienced agencies.

CHAPTER VIII

STRUCTURAL TYPES

In this chapter are presented some practical designs for the works of a hydro-electric plant. They are preceded by the nomenclature of terms herein employed, specifications of material and of methods, followed by the development of one or more designs for each separate structure of importance, with an outline of the theory of stability and adaptability from which they are evolved, and concluded with estimates of quantities and, in some cases, of cost. This treatment of structural types is largely as developed in the author's practice and as proved practical, safe, and economical for the purpose.

The dam comprises the entire structure by which the river and its valley are closed, from bank to bank, for the purpose of accumulating the water, concentrating its fall at one fixed point, and diverting the flow in the desired direction. The dam may or may not pass water over its crest; in the affirmative case it becomes a spillway, in the other a reservoir dam; generally it is a composite structure of both types, the river proper being closed by the spillway, which terminates in abutments and is flanked at one or both ends by reservoir embankments or bulkheads.

The spillway consists of the foundation and superstructure.

Article 52.—*The foundation's* functions are to prevent the passage of water below the structure and to afford rigidity of position to the superstructure. Its design is determined by the character of the material at its site, as to hardness, strength, and porosity, the height and weight of the superstructure, the maximum height of water to be ponded, and the effect of its overflow.

Foundation sites are in rock or alluvial formation.

The *primitive* rocks, formed by original solidification, fusion, or later volcanic action, are granite and sienite; they are igneous and silicious. *Granite* is composed of quartz, feldspar, and mica, with talc and hornblende as impurities; its hardness and durability are increased by the proportion of contained quartz and decreased by that of feldspar and mica; it is unstratified. *Sienite* closely resembles granite; it

consists of feldspar and hornblende with some quartz and mica; it is as hard and durable as granite.

The *transition* or *metamorphic* rocks are gneiss, sienite gneiss, greenstone, trap, and basalt; they are sedimentary, but have undergone changes due to heat, pressure, or chemical action. *Gneiss* or *mica slate* is silicious and stratified, resembling granite. *Sienite gneiss* is a stratified sienite. *Greenstone, trap,* and *basalt* consist of hornblende and feldspar and are unstratified.

The *secondary* rocks are the *sandstones,* which are formed by the solidification of disintegrated primitive rocks, being composed of grains of silicious rocks cemented together by silica, lime, and alumina. To this class also belongs *soapstone,* silicate of magnesia. The physical characteristics of sandstone vary with its density and it is generally stratified.

The *tertiary* rocks are calcareous, being formed of shells and marine animals compacted under pressure of superimposed rock or soil; to this class belong the limestones, marble, chalk, and slates. *Limestone,* carbonate of lime, varies from the hardness and density of marble to the softness and porosity of chalk. *Slate* occurs in thin strata, of which clay forms the basis.

PHYSICAL CHARACTERISTICS OF ROCKS.

Rock.	Weight per Cubic Foot.	*Crushing Strength per Square Foot.	Stratified or not.
Granite	180 lbs.	750 tons	Unstratified
Sienite	180 lbs.	750 tons	Unstratified
Gneiss	180 lbs.	700 tons	Unstratified
Sienite gneiss	180 lbs.	700 tons	Unstratified
Trap	180 lbs.	700 tons	Unstratified
Basalt	180 lbs.	700 tons	Unstratified
Greenstone	180 lbs.	700 tons	Unstratified
Sandstone	150 lbs.	600 tons	Stratified
Marble	168 lbs.	500 tons	Unstratified
Limestone (hard)	168 lbs.	500 tons	Unstratified
Slate	175 lbs.	600 tons	Stratified

Under *alluvials* are comprised gravel, sand, clay, loam, marl, and peat.

Gravel is fragmentary rock reduced by the atmosphere and water to pebbles, chiefly of quartz and of crystalline origin. *Sand* is of the same origin as gravel and consists merely of smaller particles, generally inter-

* For structural considerations 0.25 only of crushing strength here given should be accepted for safe load capacity.

mixed with gravel; it is of angular or rounded fragments as its existence is due to recent or older disintegration of the rocks. *Quicksand* consists of small rounded particles of calcareous materials which, under the influence of water, becomes like a fluid.

Clay is decomposed crystalline rock, consisting of hydrated silica or alumina, and generally contains some sand and lime; it occurs in all colors from lightest to darkest, and, according to quantity of water contained in it, is soft or stiff. *Loam* is a mixture of clay and sand, the latter predominating so far that the clay loses its coherence. *Soil* is fine earthy material mixed with more or less organic matter; *mud* is moist, soft soil; *silt* is a fine earthy sediment.

Marl is correctly classed as a tertiary formation, and is a consolidated mixture of clay and carbonate of lime which readily disintegrates when exposed to the atmosphere.

Peat is decomposed vegetable matter, spongy and containing much water near the surface.

PHYSICAL CHARACTERISTICS OF ALLUVIALS.

Material.	* Weight per Cubic Foot.	† Bearing per Square Foot.	Angle of Repose.	Coefficient of Friction.
Gravel	90–106 lbs.	2–3 tons	38°	0.78
Sand, dry and loose	90–106 lbs.	2–3 tons	28°	0.53
Sand, wet	118–129 lbs.	2–3 tons	28°	0.53
Clay, dry	119 lbs.	4–6 tons	..	
Clay, in lumps	63 lbs.	4–6 tons	45°	
Clay, damp	....	4–6 tons	45°	1.0
Clay, wet	....	4–6 tons	15°	0.27
Loam, dry, loose	72–80 lbs.	4–6 tons	35°	0.70
Loam, wet	66–68 lbs.	4–6 tons	35°	0.70
Mud	104–110 lbs.	4–6 tons	zero	0.70
Gravel and loam	....	2–3 tons	38°	0.78
Gravel and sand, dry	....	8–10 tons	45°	1.0
Loam on moist clay	....	....	45°	1.0
Loam on wet clay	....	....	17°	0.3
Clay on gravel	....	....	45°	1.0
Peat	....	....	20°	0.36

Having analyzed the character of the material at the foundation site, the following arguments and deductions should control the design.

In rock its hardness, stratification, condition and shape of surface determine the foundation.

* Compacted. † At depth beyond atmospheric influence.

In hard unstratified rock with level surface no foundation is required; the homogeneous ledge will not permit water to pass beneath its surface and it will safely carry the superstructure which is anchored and keyed to it.

In hard but stratified rock a *cut-off wall* must be constructed along the upstream side of the spillway structure in a trench excavated from the rock to a sufficient depth to pass below those strata which are less than two feet thick, its upper portion becoming part of the superstructure. The rock ledge may be of sufficient solidity to carry safely the spillway.

In soft stratified rock a *cut-off wall* is essential. The superstructure may be founded on the rock surface after the soft, disintegrated upper portions are completely removed, but an *apron* must be constructed on the downstream side of the superstructure to receive the overfall and resist its erosive force.

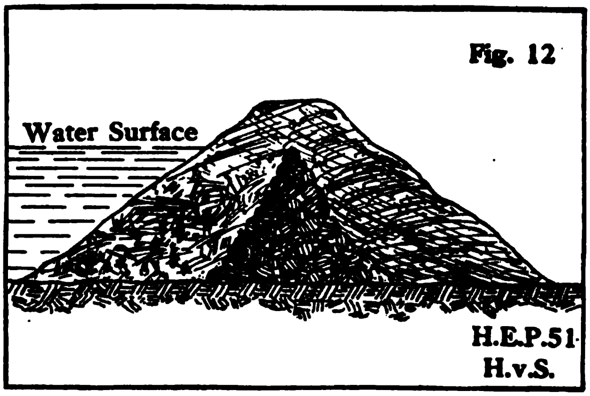
In *compact alluvial* gravel and sand, defined as hardpan, *with no interior sand strata* to a depth of one-third of the water height on the upstream side of the spillway, a *cut-off wall* is required to a depth below the river bed equal to one-fourth of the maximum water head; when the aggregate weight of the superstructure and water does not exceed two tons per square foot of its base, it may be placed directly upon the levelled, cleaned hardpan surface, but when the load exceeds this limit, *bearing piles* must be driven to support safely the superstructure, which may be placed directly upon them, or a concrete *foundation floor* may be laid, the pile tops being imbedded in it. An *apron* is required on the downstream side.

In clay with no sand strata or pockets for a depth of one-fourth of the maximum water head, *upstream cut-off walls* must be constructed to a depth of one-third of the maximum water head, and these must form a part of the foundation floor, which rests upon the bearing piles and is extended up- and downstream of the superstructure base as aprons.

In soft alluvials of clay, gravel, sand, loam, silt, or peat, upstream cut-off walls are required to a depth penetrating into impermeable material or to rock, and a pile bearing foundation with up- and downstream aprons, as will be later on described, must be constructed.

In all alluvial locations the foundation must be specially designed to provide safety against *scour* or *underwashing* and secure ample bearing capacity. Whenever sand strata exist, even at a considerable depth, at

the spillway site or upstream of it, they may rise or connect with higher strata of the same material, and eventually form channels through which the water, under increased pressure head, finds a passage beneath the spillway base, which will cause leaks and in time remove some of the mass upon which the structure rests. The only reliable safeguard under such conditions lies in securely confining so much of the mass between the cut-off and intermediate foundation walls or beams that its weight exceeds the sliding force, and to penetrate through and below this body with bearing piles which will safely maintain the superstructure, even should it, so to speak, float upon the mass enclosed between the cut-off and the foundation walls. Upon this theory the depths of the cut-off walls should be based.



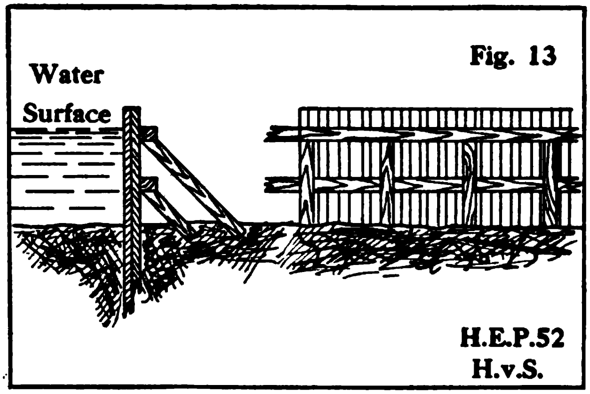
Dike.

This liability of water passing under the foundation by way of permeable strata is not confined to the river bed, but in fact is much more likely to have its origin in the banks, where the constant passage of the ground-water into the stream has formed permanent channels which, when the water is ponded above them, become readily connected by lateral channels; springs issuing from the river bank are evidences of the ground-water channels, which, however, are not always thus plainly marked. Of this subject more will be said under "Embankments."

ARTICLE 53. *Terms, Materials, and Methods.* — (A) Coffering comprises the operations and structures required to control the water during construction, to exclude it from the site.

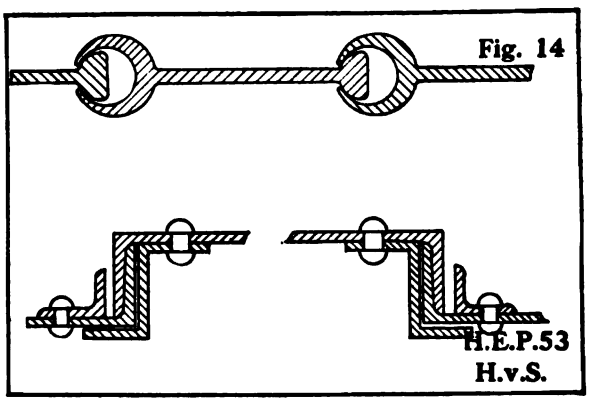
(B) *Dike*, Fig. 12, is a loosely thrown up rock-and-earth bank; its function is to exclude water from the site it encloses; it consists of a *core* of loose rock of all sizes and an earth or clay and sand *facing fill.* Loosely piled limestone and sandstone weigh 86 pounds per cubic foot; one cubic yard solid of either yields 1.75 of loose volume. A rock bank with slopes of one-half in one, no top width, and of height equal to the depth of water in which it is placed, is safe against overturning by a

factor of two, and can be made practically water-tight by placing alternate coverings of straw, hay, or manure and ashes, clay, or coarse loam against its pressure side, the water on the opposite side being maintained at a lower level while the facing is placed. The earth fill should rise 3 feet above the top of the rock core.



Timber Sheet.

(C) *Timber sheet*, Fig. 13, is a vertical curtain or wall, consisting of planks driven to overlap, of sheet piles to interlock, or of close-driven square piles for the purpose of coffering; they are effective only to enclose small areas, and must be strengthened by *waling*, which are timbers secured horizontally to them at different heights, against which inclined timbers, securely footed, are strutted. With five feet of pressure head against the timber sheet of three-inch planks, the struts must be spaced five feet. Such a sheet is made water-tight by canvas covering and earth facing. Timber sheets are driven to a depth equal to the water head against them and rise 3 feet above water surface.

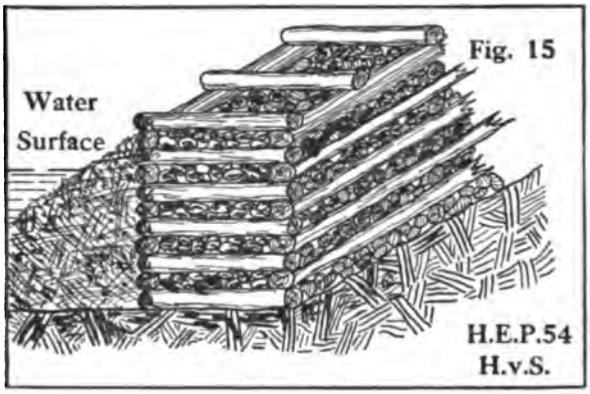


Steel Piling.

(D) *Steel sheet* consists of driven interlocking steel shapes, which are rolled of different sections from 11 to 45 pounds per foot; they are driven like timber sheets, are capable of resisting moderate pressure heads unsupported, and can be made water-tight by filling ashes against their pressure side; they can be pulled out and used again. Fig. 14 shows some of the sections now on the market.

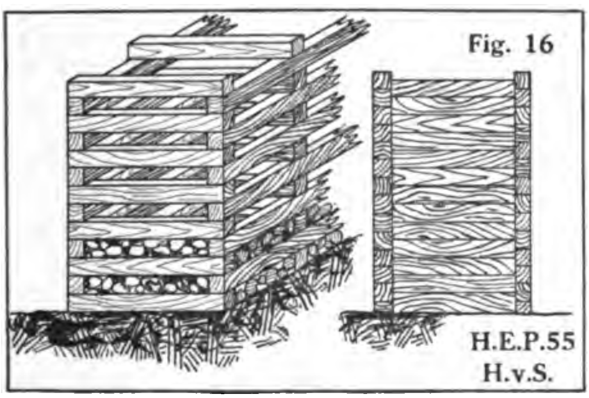
(E) *Log cribs*, Fig. 15, are constructed of round logs laid upon each other in crib fashion of alternate longitudinal and transverse streaks, the

first from five to sixteen, the second from eight to twelve feet centres; the open rectangular spaces enclosed by the logs are the *crib bays;* the logs are spiked to each other at all crossings with $\frac{3}{4}''$ round $18''$ wrought-iron drifts set in $\frac{5}{8}''$ holes; logs may be gained to secure firmer connections and to bring those of the same layer, or *streak,* to a uniform level. Log cribs are used for coffering, confining and directing flow, or retaining banks and slopes; they may be framed in place, the second streak being formed into an open log floor and the bays filled with gravel or loose rock as the framing progresses, or they may be floated light into position and there loaded and sunk; they can be made water-tight by planking or boarding, or by diking. To resist overturning, with a factor of two, their interior width should equal 0.75 of the water height and their rock-filled height should rise three feet above the water surface.



Log Crib.

(F) *Timber cribs,* Fig. 16, are built of dimension timber in crib fashion similar to log cribs, or of solid timber walls, and are filled with rock, gravel, or sand and made water-tight by canvas covering and puddle placed along their footing on the pressure side. To resist overturning, with a factor of two, the width of timber crib should be 0.66 of the water height and the filled height should equal that of the water.

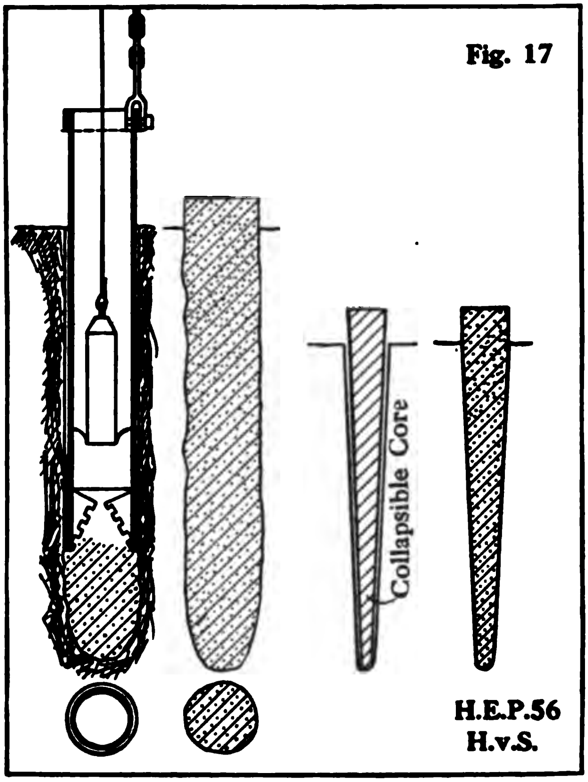


Timber Crib.

(G) *Timber.—Boards* are one inch thick, and sawn; *planking* is two or three inches thick, sawn; *scantling* is $2'' \times 3''$ or $4''$, sawn. *Dimension* comprises all sawn or hewn sticks of rectangular or square section ex-

ceeding three inches in thickness; *ft. b. m.* is the abbreviation for feet board measure, the unit of which is one square foot one inch thick. *Round timber measure* is in cubic feet = length $\times \frac{C^2}{13}$; C is the circumference of the log in feet.

(H) *Bearing piles* are straight logs 16 feet and longer, 8″ in diameter at the small end under the bark; they must be of one year's cut and sound and are barked. They are *driven to refusal* when they do not sink more than 0.5 inch under a free 20-feet drop of a 2000-pound hammer.



Concrete Piles.

The theoretical *bearing capacity* of timber piles equals cube root of fall of hammer in feet x weight of hammer in pounds x constant 0.023, divided by constant 1 + sinking distance per stroke in inches, which for "driven to refusal" conditions are $\sqrt[3]{20} \times 2000 \times 0.023 \div 1.5 = 64$ tons. The actual loading should not exceed one-half of theoretical when the pile stands in gravel, clay, and sand; one-fourth of theoretical when the pile stands in clay and sand; one-tenth of theoretical when the pile stands in mud. To drive bearing piles in compacted sand is difficult and generally requires clearing by water jet.

(I) *Concrete piles* are constructed by different processes. In Fig. 17 a collapsible steel core inside of a steel plate shell is driven, the core is withdrawn and the shell filled with concrete. In the other type a steel form fitted with a bucket point is driven to the required depth, some concrete is then lowered into it and the shell is pulled up two feet, by which operation the bucket point opens and permits the concrete to drop to the bottom and it is there rammed in place; lowering more concrete,

pulling shell two feet, and ramming the concrete are repeated until the pile is completed. These piles can be re-enforced by imbedding steel rods in the concrete.

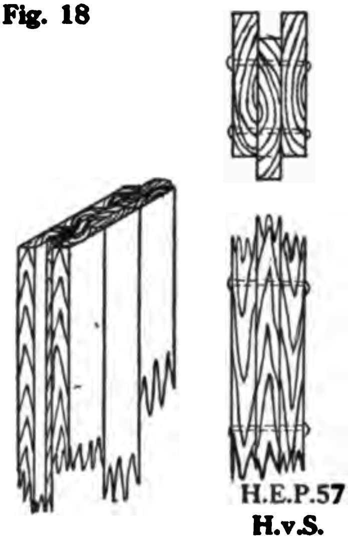
(J) *Cut-off wall* intercepts flow and seepage below the surface; it may be of puddle, timber or steel curtain, or a concrete wall.

(K) *Core wall* serves the same purpose as the cut-off in the interior of earth embankments.

(L) *Puddle* is a plastic mass of clay, small gravel, and coarse sand in the proportions of 5 to 3 to 2, compacted in a confined space.

(M) *Timber curtain* serves the purpose of cut-off or core wall, and consists of triple-lap sheet piles, Fig. 18, which are constructed of three planks placed on face sides, centre plank overlapping, thus forming tongue and groove on opposite edges; planks are secured by wrought-iron boat spikes one inch longer than thickness of pile, spikes are driven from opposite sides of pile, points are clinched.

Fig. 18



Triple-lap Sheet Pile.

Piles up to 8 feet long are 6 inches thick; piles up to 16 feet long are 9 inches thick; piles up to 24 feet long are 12 inches thick. One end is scarf pointed so that, when driven, it crowds toward preceding pile, forcing the tongue into the groove. These piles should be driven with a light hammer and a low drop and be guided all the way down; the timber for them should be pine or hemlock, sound, of uniform thickness, and preferably edged.

(N) *Steel curtain* is constructed of steel sheet pile sections described under D.

(O) *Concrete* herein considered consists of *true Portland cement* mortar and hard broken stone or gravel; the mortar is composed of one part of cement and two or three parts of sand by weight and of sufficient volume to fill completely the voids in the aggregate. A 1 : 3 : 6 mixture is designated as x concrete, a 1 : 2 : 5 mixture is called xx concrete. *The cement and sand* should conform to the standard specifications of the American Society of Civil Engineers; the aggregate is sized from one-quarter inch to two and one-half inches.

The mixing is by hand or in *batch* mixers as follows: the cement and sand are dry mixed; water is added and the mortar mixed, the aggregate is added, thoroughly washed and wet, and the concrete is mixed to a wet but not flowing mass,—that is, water should not stand on its surface nor ooze out of it during handling or transporting.

The concrete is placed in a manner to avoid disintegration of the mixture when dropped from cars, barrows, or shovels; the height of its fall should be restricted, and the inclination of chutes must not be steep.

When concrete is placed on rock, the surface should be cleaned of all vegetable and earthy matter and drenched with water, and smooth rock surfaces should be scarred.

Unfinished joints are to be treated in the same manner as rock faces before new concrete is added, and the maximum length of unfinished joints on the same plane should not exceed ten feet.

Finished concrete should be covered, if practicable, with a layer of wet sand, or otherwise shaded from the sun and kept damp, for 48 hours.

Such concrete may be expected to develop in six months the following characteristics, expressed in pounds per square inch:

TABLE 4.—CONCRETE CHARACTERISTICS.

		X Concrete.	XX Concrete.
Modulus of elasticity	E_c =	3,000,000	2,400,000
Compressive strength	f_c =	2,000	2,400
Tensile strength	f_t =	200	200
Shearing strength	C =	300	300
Expansion	=	0.000006 per degree F.	
Adhesion to rust-free steel	=	600	

Working stresses should be taken at 0.25 of above.

Cyclopean concrete has imbedded in the concrete mass solid stones of any size, each stone being surrounded by not less than a twelve-inch wall of concrete, and no stones being placed closer to the finished surfaces of the structure than two feet. *Monolithic concrete* is a solid mass of concrete. *Block concrete* consists of shaped concrete units laid in the manner of ashlar or coursed masonry in Portland cement mortar.

Forms are shapes of boards, planking, or metal walls in which concrete is moulded into blocks, walls, partitions, arches, or beams; the interior sides of the forms are smooth and oiled. Forms should not be removed until the concrete is fully set, generally 48 hours.

Concrete steel, or reinforced concrete, is a structure in which the

imbedded steel increases the strength of the entire section; or in which the proportions of the steel to the concrete area, and the location of the steel with reference to the stresses, are determined to secure the action of both as a unit. The random placing of steel members in a concrete mass, or the lining or supporting of any side or face of a concrete section with steel, is not of the class herein considered. The characteristics of reinforcing steel are taken in pounds per square inch as follows:

TABLE 5.—REINFORCING STEEL CHARACTERISTICS.

Modulus of elasticity	Es =	29,000,000
Ultimate strength	fs =	64,000
Elastic limit	F =	50,000
Expansion	=	0.0000065 per degree of F.

The concrete steel designs herein used are based upon the concrete and steel characteristics given in *Tables* 7 *and* 8, and are according to formulæ of Mr. A. L. Johnson, M. Am. Soc. C. E.

TABLE 6.—CONCRETE STEEL BEAMS.

	X Concrete.	XX Concrete.
Moment of ultimate resistance	= $3620\ t^2$	$5505\ t^2$;
Area of steel in width of sec.	q = 0.077 t	0.132 t;

t is depth of the beam.

Values of t and q for beams to resist various bending moments in accordance with these formulæ are given in Table 7. Many different kinds and shapes of reinforcing steel rods are used in such structures, but the above formulæ apply generally and differ only with the change of concrete and steel characteristics.

TABLE 7.—VALUES FOR CONCRETE STEEL BEAM DESIGNS.

M = ultimate bending moment of external forces in 1000 inch pounds; t = depth of beam in inches; q = square inches of steel in one foot width of beam. Steel is placed 0.9 t from compression face of the beam.

	X Concrete.		XX Concrete.	
M.	t	q	t	q
100	5.27	0.408	4.27	0.562
150	6.45	0.500	5.22	0.689
200	7.45	0.576	6.02	0.795
250	8.32	0.644	6.74	0.889
300	9.12	0.706	7.38	0.975
350	9.85	0.762	7.93	1.050
400	10.52	0.816	8.52	1.125
450	11.15	0.861	9.05	1.193
500	11.73	0.910	9.53	1.258

M.	X Concrete. t	X Concrete. q	XX Concrete. t	XX Concrete. q
550	12.38	0.956	10.00	1.320
600	12.90	0.998	10.44	1.380
650	13.40	1.040	10.84	1.435
700	13.92	1.078	11.29	1.486
750	14.40	1.113	11.68	1.540
800	14.88	1.151	12.02	1.588
850	15.34	1.188	12.41	1.640
900	15.80	1.222	12.79	1.686
950	16.25	1.258	13.11	1.735
1,000	16.68	1.289	13.49	1.780
1,500	20.40	1.580	16.50	2.180
2,000	23.50	1.812	19.05	2.520
2,500	26.30	2.038	21.30	2.810
3,000	28.80	2.230	23.35	3.075
3,500	31.15	2.410	25.20	3.325
4,000	33.25	2.578	26.90	3.560
4,500	35.25	2.730	28.59	3.780
5,000	37.20	2.880	30.10	3.980
5,500	39.10	3.025	31.60	4.180
6,000	40.80	3.160	33.05	4.360
6,500	42.50	3.285	34.39	4.530
7,000	44.00	3.410	35.65	4.700
7,500	45.60	3.530	36.90	4.870
8,000	47.00	3.640	38.10	5.030
8,500	48.55	3.760	39.30	5.190
9,000	49.90	3.860	40.40	5.340
9,500	50.25	3.965	41.50	5.585
10,000	52.70	4.075	42.60	5.620

(P) *Breakwater* consists of log cribs placed ten to sixteen feet centres, the intervening spaces being closed by planks or timbers placed in an inclined position, ends resting on the stream bed and tops against longitudinal logs secured to the top of cribs; its purpose is to check swiftly flowing water.

(Q) *Sheet pile dike*, Fig. 19, is employed for coffering service; it consists of two parallel sheet pile curtains from five to fifteen feet apart, the area between them being filled with puddle, and the pressure side, or both sides, being covered with riprap and facing fill.

(R) *Riprap* is loose rock thrown up against a bank, wall, or curtain, to break the force of flowing water or resist its pressure.

(S) *Steel pile dike* is a structure similar to the sheet pile dike, the curtain being of steel sheet piles.

(T) *Paving* consists of large flat stones placed by hand on faces or edges, interstices being filled with stone chips or spalls, on slopes of earth banks or surfaces, to prevent erosion.

View 1
Breakwater

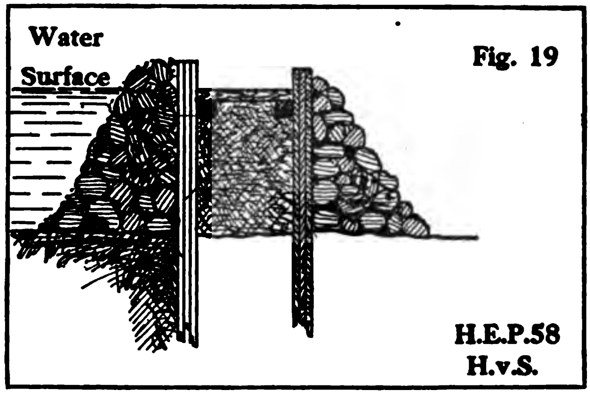
View 2
Timber Crib

View 3
Log Crib Coffer

View 4
Sheet Pile Dike

ARTICLE 54.—*Coffering* is the first operation preparatory to the construction of any part of the dam, in order to exclude the water from the construction site. It requires judgment born of experience to confine the means employed, which will accomplish this, to devices of temporary character and economical cost; their failure will generally cause damages which would cover the cost of the most permanent works. Depth of water, velocity of flow, river-bed material, area to be coffered, and possibility of exposure to flood rises are the conditions to be weighed in determining the recommendable programme.

In a rock bed and shallow water where the velocity does not exceed three feet per second, a dike (53, B) answers the purpose until its required section exceeds the cost of a log crib (53, E), which latter must be chosen for swift water. *In rapids* a breakwater (53, P) should first be constructed, such as is seen in View 1, which was erected in the Sault Rapids at the foot of Lake Superior, being 300 feet long in ten-feet-deep water with velocity of twelve feet per second. Rock is not always the most economical filling for cribs, as, for example, in the case of the location just referred to, View 2, where the construction site was coffered by a timber crib filled with sand which was pumped in. *In gravel and clay beds* a dike answers until its cost exceeds that of a log crib, as the one shown in View 3, which was 250 feet long, placed in ten feet of water and exposed to considerable wave action.



Sheet Pile Dike.

In clay and sand and depth of water exceeding ten feet, a sheet pile dike (53, Q) will prove the most serviceable, such as is shown in View 4, which was placed by the author in twenty feet of water, was two thousand feet long, and consisted of two timber curtains, fifteen feet centres, braced and strutted, filled with puddle, riprapped and banked; it remained in service during a period of three years and developed no leaks. When water is shallow, timber or steel sheets (53, C and D) may answer; two examples of the latter, constructed of different type of steel-sheet piles,

are shown in Views 5 and 6. *In compacted sand* a steel sheet may prove the only practicable coffer up to ten feet depth of water; when deeper, a steel pile dike may be the proper solution; timber sheets cannot be driven successfully in this class of material. In soft formations and water depth of five feet or less, timber sheets will answer; in greater depths the sheet pile dike is preferable to cribs, as it will be difficult to find firm footing for the latter.

In all coffer operations it is essential to provide, and constantly maintain, ready means to add promptly filling, riprap, and facing material to the coffering structures, and further to provide and keep in commission a sufficient *pumping plant* to remove all water accumulating from leaks and seepage which is collected in a *pump sump* conveniently located.

TABLE 8.—QUANTITIES REQUIRED FOR DIFFERENT COFFER STRUCTURES IN 10-FEET LENGTHS, AS PER SECTIONS SHOWN IN FIGS. 12 TO 19.

Depth,	Sheets.		Dike.		Sheet Pile Dike.			Log Crib.			Timber Crib.		
Water, ft.	Timber, ft. b.m.	Steel, lbs.	Cub. yds. Rock.	Puddle.	Timber, ft. b.m.	Puddle, cub. yds.	Riprap, cub. yds.	Logs, lin. ft.	Drifts, lbs.	Rockfill, cub. yds.	Timber-Drifts, ft. b.m.	Spikes, lbs.	Sandfill, cub. yds.
5..	1230	1430	5	45	2400	10	12	125	150	7	1750	175	5.5
6..	1420	1650	7	54	2800	13.3	15	155	170	11.7	2175	200	9
7..	1630	1870	9	64	3200	17	19	174	190	15.5	2475	225	13
8..	1860	2090	12	75	3600	21.6	23	198	205	20	2825	250	17
9..	1880	2310	15	88	4000	27	28	220	220	25	3150	270	21
10..	2300	2530	18	100	4400	32	33	250	230	30.5	3600	290	26
11..	....	....	22	113	4700	37.8	37	275	245	35.5	4000	315	31
12..	....	....	27	126	5000	44.4	42	294	260	43	4275	335	36.5
13..	....	....	31	141	5450	51	47	317	275	52	4625	360	42
14..	....	....	36	154	5800	58.5	53	352	290	61	5150	380	48
15..	....	....	42	171	6200	66.6	60	375	305	71	5525	410	55
16..	....	....	..	...	6700	75	66	400	320	80	5900	430	65
17..	....	....	..	...	7100	83.7	74	425	335	89	6325	460	75
18..	....	....	..	...	7500	93	82	450	350	98	6700	480	85
19..	....	....	..	...	7900	102.8	90	475	360	107	7100	500	95
20..	....	....	..	...	8400	112.4	98	515	380	117	7700	530	105

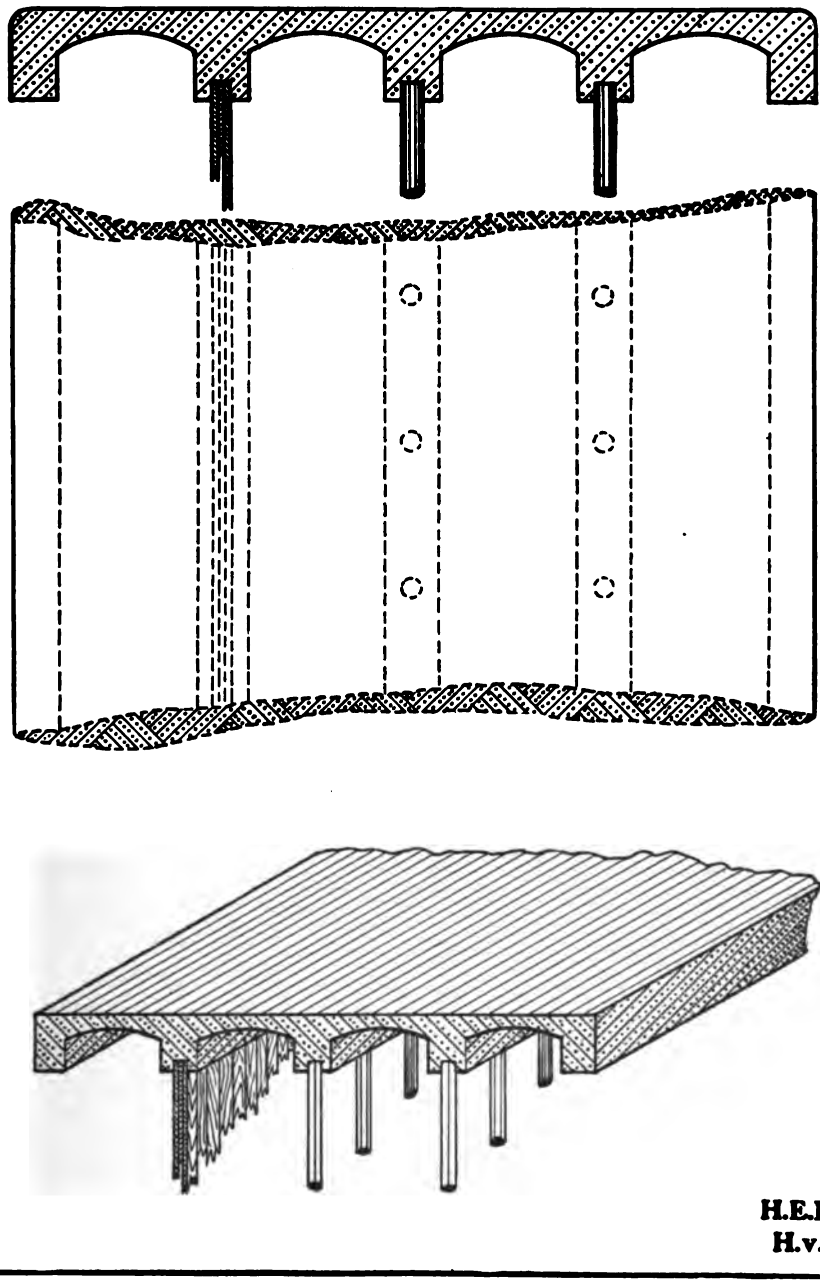
ARTICLE 55.—*A foundation design* for a thirty-feet high spillway in alluvial location, based upon the theories outlined in Art. 52, is shown in *Plan* 18. The structure consists of a cut-off wall, bearing piles, floor walls or beams, and floor. *The cut-off* may be a driven timber or steel curtain, provided the river bed material is such that these can be driven in a manner guaranteeing an unbroken, solid curtain. Timber sheet piles do not secure this result if driven through coarse gravel, or if boulders are encountered, as these will deflect the driving point and force the tongue out of the groove, leaving openings between sheet piles, and thus

VIEW 5.—Steel Pile Coffer.
U. S. Steel Pile.

VIEW 6.—Steel Pile Coffer.
Friestedt Channel Bar.

Plan 18

Foundation design



H.E.P.65
H.v.S.

defeat the purpose for which they are employed. Steel piles readily penetrate gravel without deflection, but not so when large boulders are struck, as their interlocking parts may then be ruptured and thus also leave openings. These defects are not readily detected during the process of driving, and therefore the decision as to adaptability of driven curtains for cut-off service must be solely based upon the correct diagnosis of the material, as all other considerations are insignificant in importance when compared to the results expected from the foundation cut-off wall.

A driven curtain presents the double advantage of economy and expedition, as it can be frequently driven before any coffering operations are started and may in part also serve to aid in coffering and, when of timber, it forms the most economical cut-off wall type. The tendency, therefore, predominates to rely upon this form; but these influences must not be permitted to outweigh the sound judgment of the engineer; when the presence of numerous and large boulders has been fully established, a driven curtain should under no conditions be relied upon.

If the material favors the use of a driven curtain, the piles must be constructed and driven with the greatest care, and this operation, above all others, demands the most rigid supervision on the part of the engineer.

When a timber curtain cannot be used, it is not always likely that the steel curtain solves the problem: the very conditions which prohibit a timber curtain form difficulties in the construction of a perfect steel curtain. The cost should be estimated, as it will come close to, and perhaps exceed, that of a concrete wall, as will appear in a comparative estimate of quantities for various cut-off structures at the end of this article. Steel curtains are described in 53, D, and it is just as essential to observe every detail of their construction as with timber curtains. When carefully placed they form an effective cut-off wall; but they are perishable, nor can their life be prolonged by any protective coating, as this would be more or less removed during driving. Steel which had been imbedded in wet clay and sand for six years was found reduced in section nearly one-half by corrosion when recovered by the author.

When driven curtains are not adapted for cut-off wall, *trenching* must be resorted to and the question of cut-off type further examined. During trenching much valuable information is gained as to the exact subsurface formation, and advance borings from different levels will greatly add to this. When the required depth is reached, a close estimate

can be made as to which is the most economical cut-off, a timber curtain or concrete wall; both will give equally satisfactory results as cut-offs, as the timber sheet piles can now be placed with perfect interlocking, and their durability is guaranteed by their constant saturated condition. It becomes then solely a question of cost, which, with low-price concrete and high cost of timber, may be in favor of the former or, with values reversed, the latter may be more economical.

The placing of timber curtains in the trench needs no further detailing nor that of the concrete wall. Trenches should be three feet wide to ten feet depth, four to twenty, five to thirty, and six to forty feet depth, in order to provide room for shoring and handling of material. Concrete cut-off walls may be uniformly three feet thick; the remaining trench-space is refilled with puddle (53, L).

TABLE 9.—QUANTITIES FOR CUT-OFF WALLS IN TEN-FEET LENGTHS.

Depth in feet.	Driven Curtains. Timber, ft. b. m.	Spikes, lbs.	Steel, lbs.	Trenched Curtains. Trenching, cub. yds.	Puddle, cub. yds.	Concrete Wall. Concrete, cub. yds.	Puddle, cub. yds.
6	600	60	660	7	5	7	
8	800	80	880	9	6	9	
10	1,000	100	1,100	11	7	11	
12	1,200	120	3,000	18	13.5	13	5
14	1,400	140	3,500	21	15	15.5	5.5
16	1,600	160	4,000	24	18	18	6
18	1,800	180	4,500	27	20	20	7
20	2,000	200	5,000	30	23	22	8
22	2,200	220	7,700	41	33	24	15
24	2,400	240	8,400	44	35	27	17
26	2,600	260	9,100	48	38	29	19
28	2,800	280	9,800	52	41	31	21
30	3,000	300	10,500	55	49	33	22
32	3,200	320	13,400	71	59	35.5	35.5
34	3,400	340	14,280	75	62	38	37
36	3,600	360	15,120	80	67	40	40
38	3,800	380	15,960	85	71	42	43
40	4,000	400	16,800	90	75	45	45

ARTICLE 56. *Foundation.*—When a sufficient area of the foundation site is safely coffered and the water removed by draining or pumping or both, the site is prepared by being levelled to a uniform plane, and all vegetable matter, roots, etc., removed; then the bearing piles are driven as per Plan 18 and, as specified in 53, H, to refusal. Piles must be driven with care; brooming of tops should be avoided; in hard driving timber *followers* should be used. In order to guard against the driving of piles

which prove too short, or prevent waste by using those which are too long, advance borings should be made by which the character of the material and thus its penetrability can be determined with certainty. In very soft soils every other longitudinal pile row should be driven at a batter leading downstream, by which additional resistance to sliding of structure will be secured, since a downstream movement would have to be accompanied by a rising upward of pile tops and thus involve lifting of the foundation walls in which they are imbedded and of the superstructure. All piles should be driven before any of the concrete is placed, to avoid possible rupture caused by vibration due to pile driving. Pile tops are cut off on a uniform horizontal plane being that of the bottom of foundation floor.

The trenching for floor walls should begin at the upstream cut-off wall, if it is a trenched structure, and only so much trench is opened as can be kept entirely free of water by pumping; the wall concrete is of x mixture and is compacted in the trench on both sides of the curtain top and covering the same. If the superstructure is to connect with the cut-off wall, as is generally the case, re-enforcing steel rods are imbedded in this cut-off concrete. The other floor walls or beams are placed likewise successively, steel rods or dowels being imbedded in them two feet centres. This concrete should not be too wet, as the soil itself will contain more or less water, and as all of it should be given considerable ramming. The top surfaces of floor walls must be left as rough as possible.

The foundation floor consists, as shown in Plan 18, of separate arches, longitudinal to the floor, the walls forming the skewbacks. Beginning at the upstream end, the soil between cut-off and first upstream wall is trimmed away to the arch form, and, the section being kept entirely free of water, the respective floor arch is laid of x concrete, the wall tops being first cleaned, roughened, and thoroughly wetted, and the concrete rammed into place. Along the top of each floor wall a groove is left free of concrete, one foot wide and deep, to form a connecting key for the superstructure, and, when the latter consists of transverse buttresses or partitions, similar grooves are formed in the floor transversely. In this manner the entire foundation floor is constructed. *Quantities* in such foundations for spillways of different heights are given on Diagram 12.

ARTICLE 57. *Superstructure.*—The subjects to be considered for the selection of a recommendable type are the height, length, section, character of material in river bed, flood volume to be controlled, fluctuations

of fall, floatage and ice, legal requirements, location of the power station, availability of structural material and skilled labor, and of transportation facilities to the site.

The height is fixed, generally speaking, by the available power fall and the depth of water in the river at the site. The fall is to be based upon conditions prevailing when the volume of flow is that which is to be diverted, and is represented by the total fall in the stream over the reach to be controlled *less the slope* which will prevail in the upper pool, proper weight being given to the flood conditions. The slope or *back-swell* is the fall in the pond above the dam which creates the flow; this is most readily determined from the application and solution of the flow formula for a sufficient number of stream sections in the length which will be covered by the upper pool. For this purpose a contour plan of the stream valley, to the height to which the pond level is to be raised, must be available, and, assuming then section B B at some point above the dam site, the cross-section is plotted as on Plan 19, from which the

wetted perimeter is scaled .... P = feet,
the flow area to the future pond level is computed .... A = square feet,
and the hydraulic radius is found from A ÷ P. .... R =
Assuming the flow volume which is to be diverted
when no water passes over the spillway as........... Q = cub. sec. ft.,
the velocity with which it passes through the
section .... B B is V = ft. per sec.
The flow formula best adapted to solve the problem
is that of M. H. Bazin:........................... $RS = (a + \frac{b}{R}) V^2$,
in which

R is the hydraulic radius,
S is the slope of the water surface,
a and b are constants expressive of the retarding influence of P, and
V is the velocity of the flow.

All the factors in this formula are known with the exception of S, and this is the value which is sought, as from it the fall in the pool's surface, which causes the flow toward the dam, is found.

"S" can be taken from Diagrams 20 to 25 for the different river-bed characteristics, whether in rock, gravel and sand, or clay and sand.

According to the length of the pond and the uniformity of its prism, S is determined for the centre section of lengths of 1000 feet or more, or of such as represent approximate similarity of channel in area and shape, and the slope found for it is credited to that reach, and thus the aggregate slope or swell is determined for the entire pool until its upper terminal is reached, which will be detected by the rapid increase of S.

Example.—Length of pond deduced from the horizontal plane of the produced spillway crest level is 12 miles; the flow to be diverted, none passing over the spillway, is 2600 cubic second feet, the height of the dam crest above the river surface with this flow is 20 feet; the length of the spillway is 200 feet. For the first 5000 feet above the dam the pond lies between uniformly sloping banks and is of approximately uniform cross areas, the perimeter is of gravel and sand formation; section A is therefore taken midway, or 2500 feet above the dam, where

$$P = 243, \quad A = 3466, \quad R = 14.2, \quad \text{and} \quad V = 0.75,$$

$$S = \left(0.000122 + \frac{0.0007}{14.2}\right)\frac{0.75^2}{14.2} = 0.000007;$$

the total fall in this reach of 5000 feet is therefore = 0.035 ft.

The next 3000 feet of the pond is of smaller flow area, the pond is narrower, the material the same; a section B is taken midway, or 6500 feet above the dam, where

$$P = 161, \quad A = 2500, \quad R = 15.5, \quad \text{and} \quad V = 1.0,$$

$$S = 0.000011, \text{ the fall in this reach} = 0.033 \text{ ft.}$$

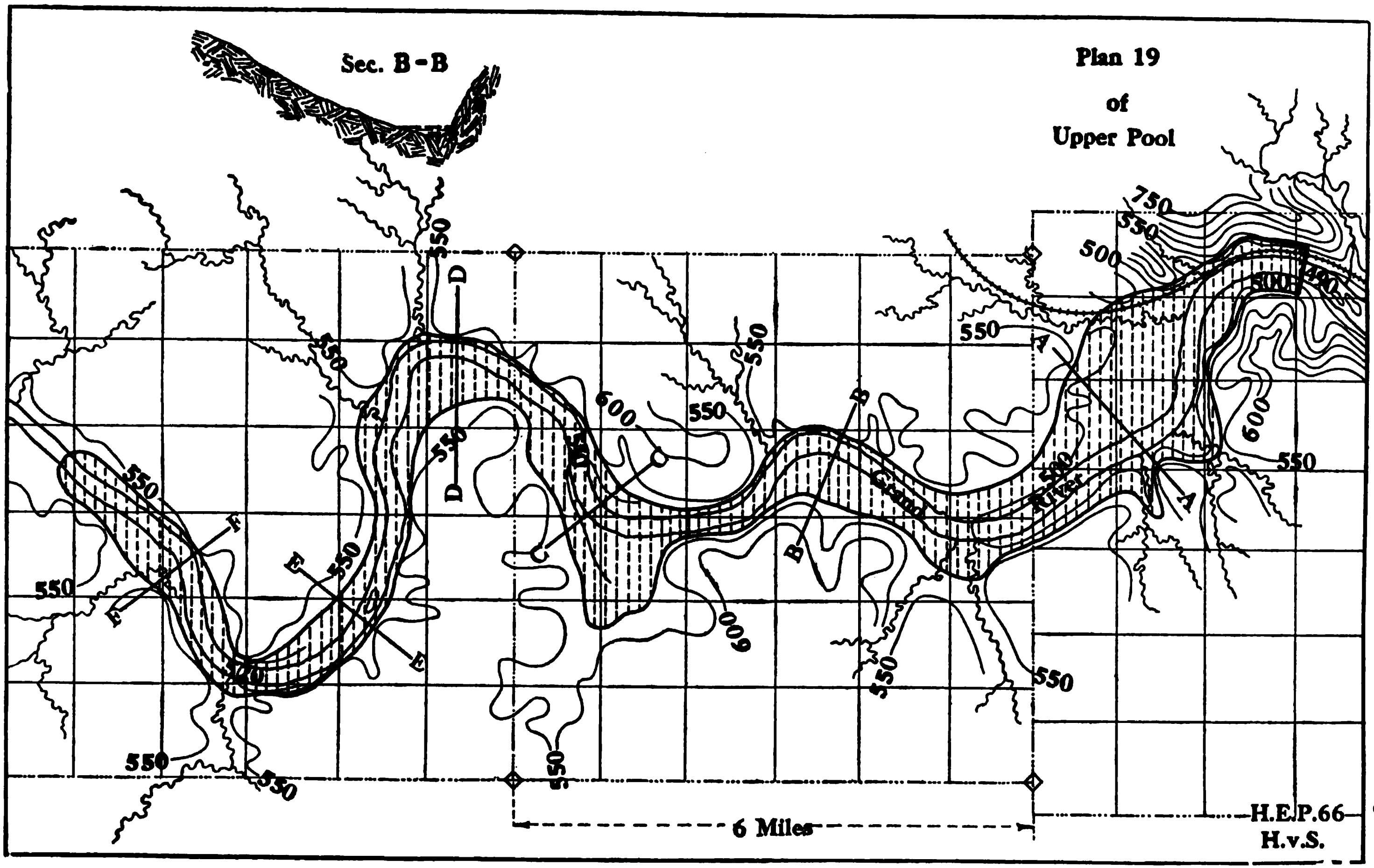
Then the pond widens again for the next 4000 feet, and at mid-section C, or 10,000 feet above the dam,

$$P = 240, \quad A = 2780, \quad R = 11.6, \quad \text{and} \quad V = 1.0,$$

$$S = 0.000009, \text{ the fall in this reach therefore} = 0.036 \text{ ft.}$$

Thus for 12,000 feet of the pond the total fall or swell = 0.104 ft.

In this manner the process is continued to the head of the pond, or about twelve miles in this case. From this point up the original stream condi-



Plan 19
of
Upper Pool

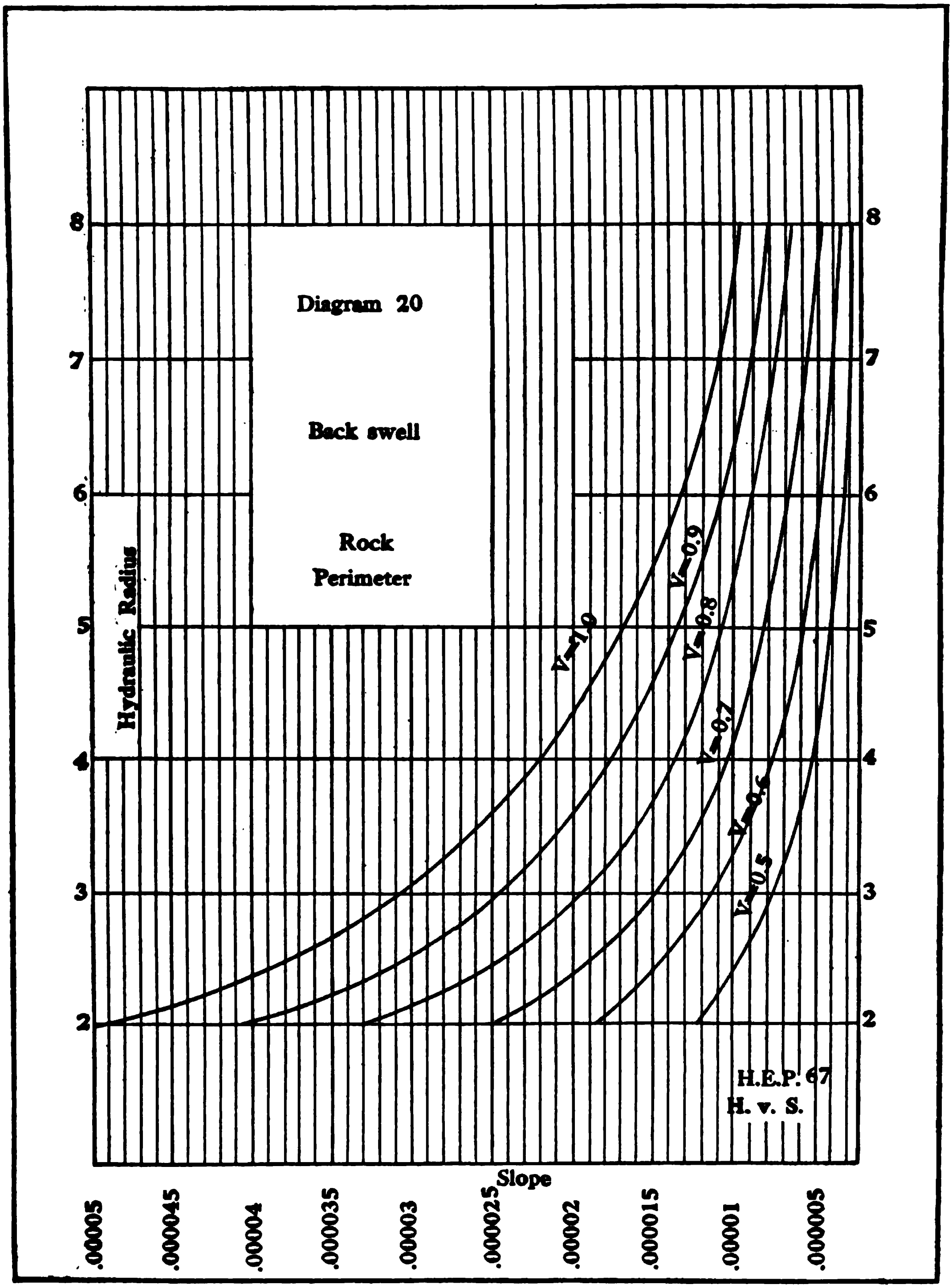


Diagram 20
Back swell
Rock
Perimeter
Hydraulic Radius
V=1.0
V=0.9
V=0.8
V=0.7
V=0.6
V=0.5
2
3
4
5
6
7
8
H.E.P. 67
H. v. S.
Slope
.00005
.000045
.00004
.000035
.00003
.000025
.00002
.000015
.00001
.000005

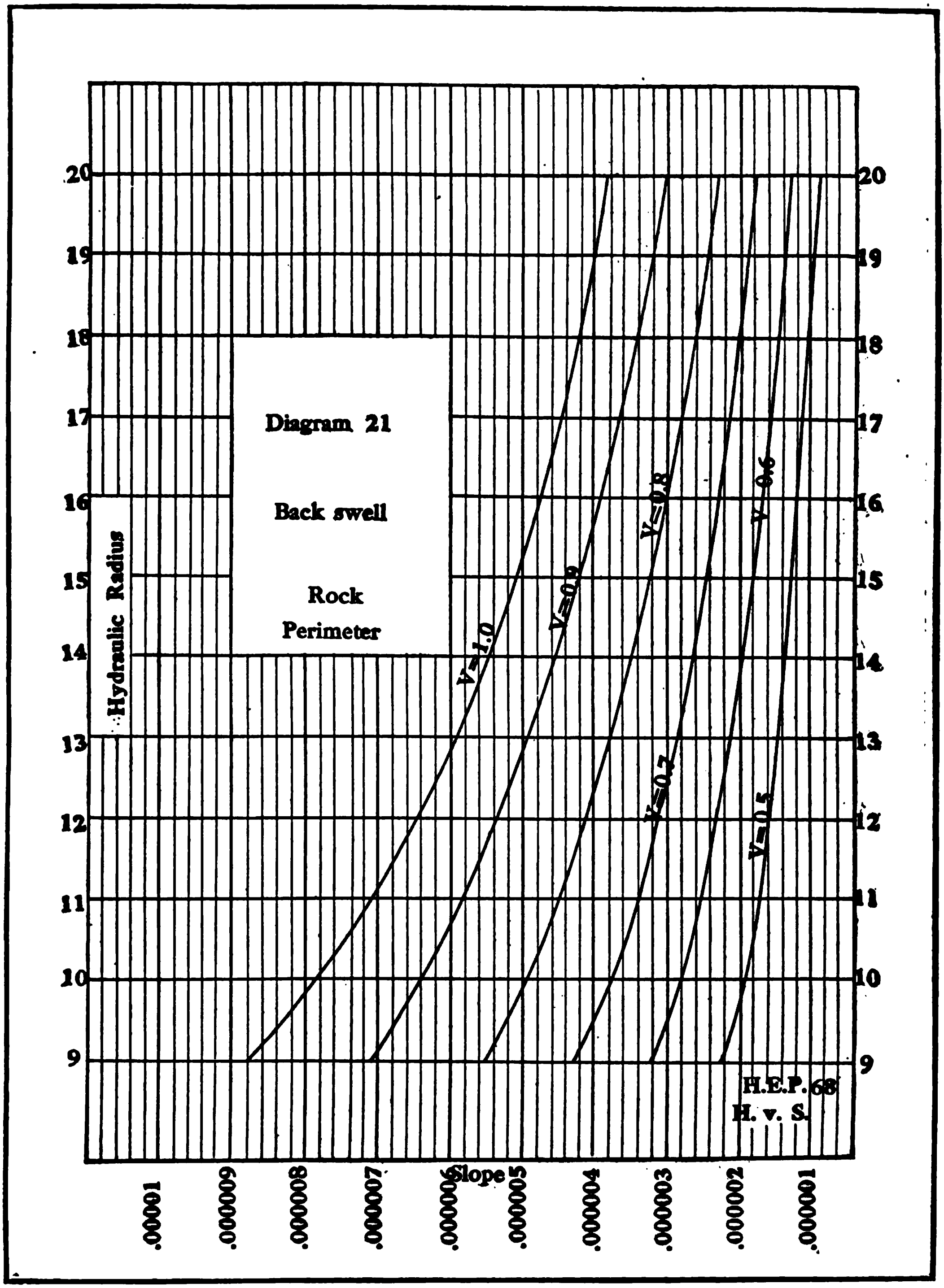


Diagram 21
Back swell
Rock
Perimeter
Hydraulic Radius
Slope
V=1.0
V=0.9
V=0.8
V=0.7
V=0.6
V=0.5
H.E.P.68
H. v. S.
20
19
18
17
16
15
14
13
12
11
10
9
.00001
.000009
.000008
.000007
.000006
.000005
.000004
.000003
.000002
.000001

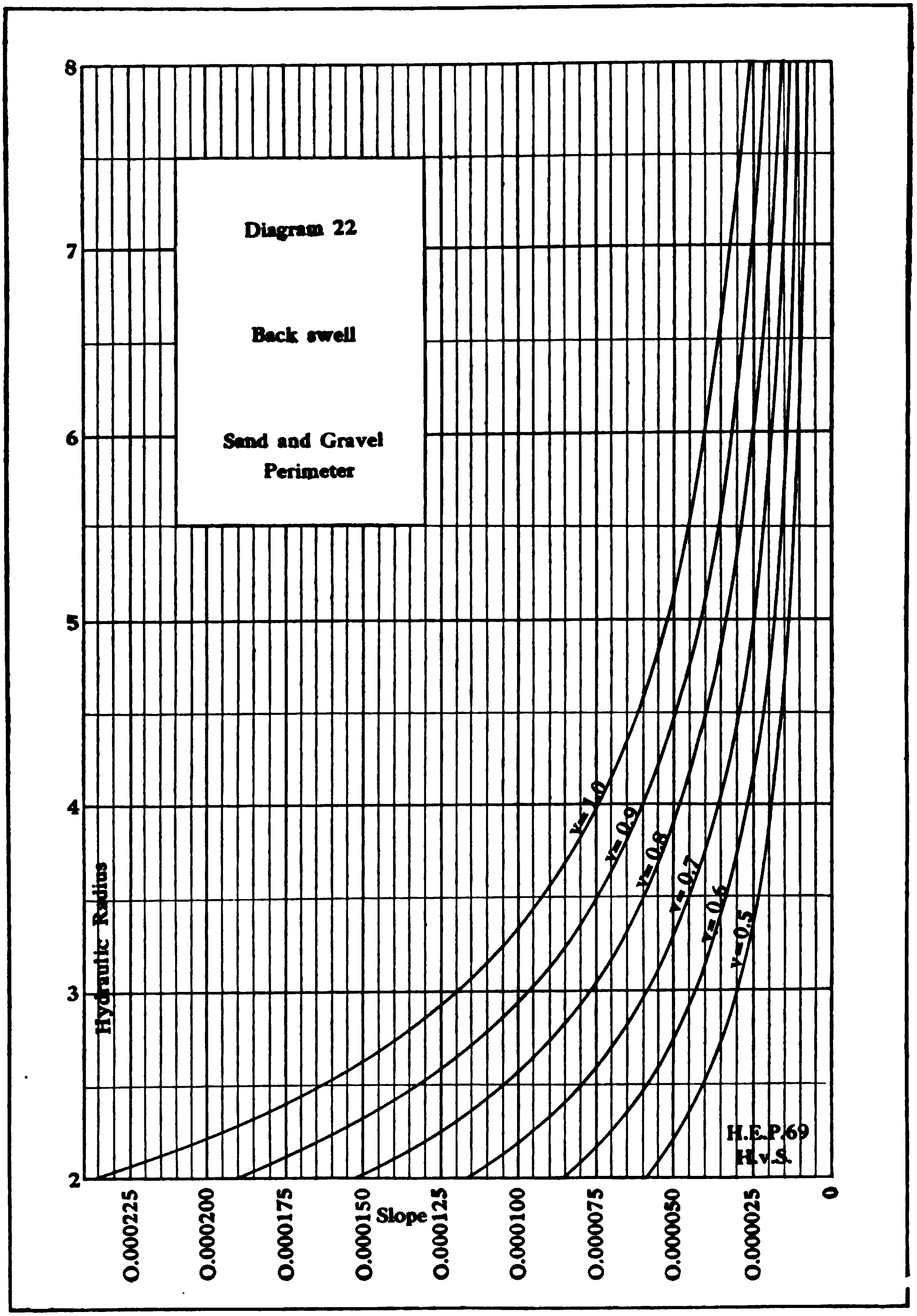


Diagram 22
Back swell
Sand and Gravel
Perimeter
Hydraulic Radius
8
7
6
5
4
3
2
Slope
0.000225
0.000200
0.000175
0.000150
0.000125
0.000100
0.000075
0.000050
0.000025
0
v=1.0
v=0.9
v=0.8
v=0.7
v=0.6
v=0.5
H.E.P.69
H.v.S.

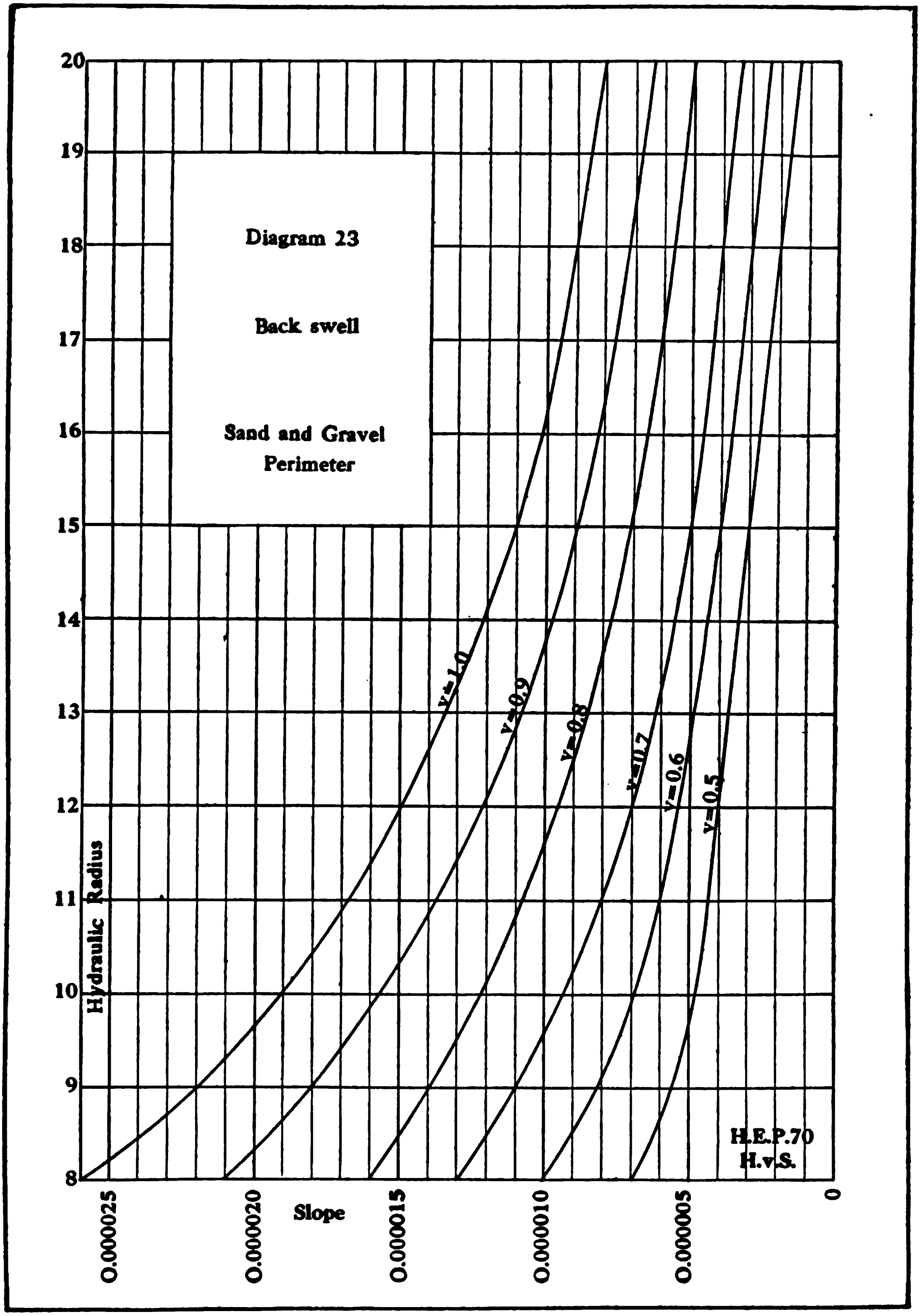


Diagram 23
Back swell
Sand and Gravel
Perimeter
Hydraulic Radius
Slope
v=1.0
v=0.9
v=0.8
v=0.7
v=0.6
v=0.5
H.E.P.70
H.v.S.
20
19
18
17
16
15
14
13
12
11
10
9
8
0.000025
0.000020
0.000015
0.000010
0.000005
0

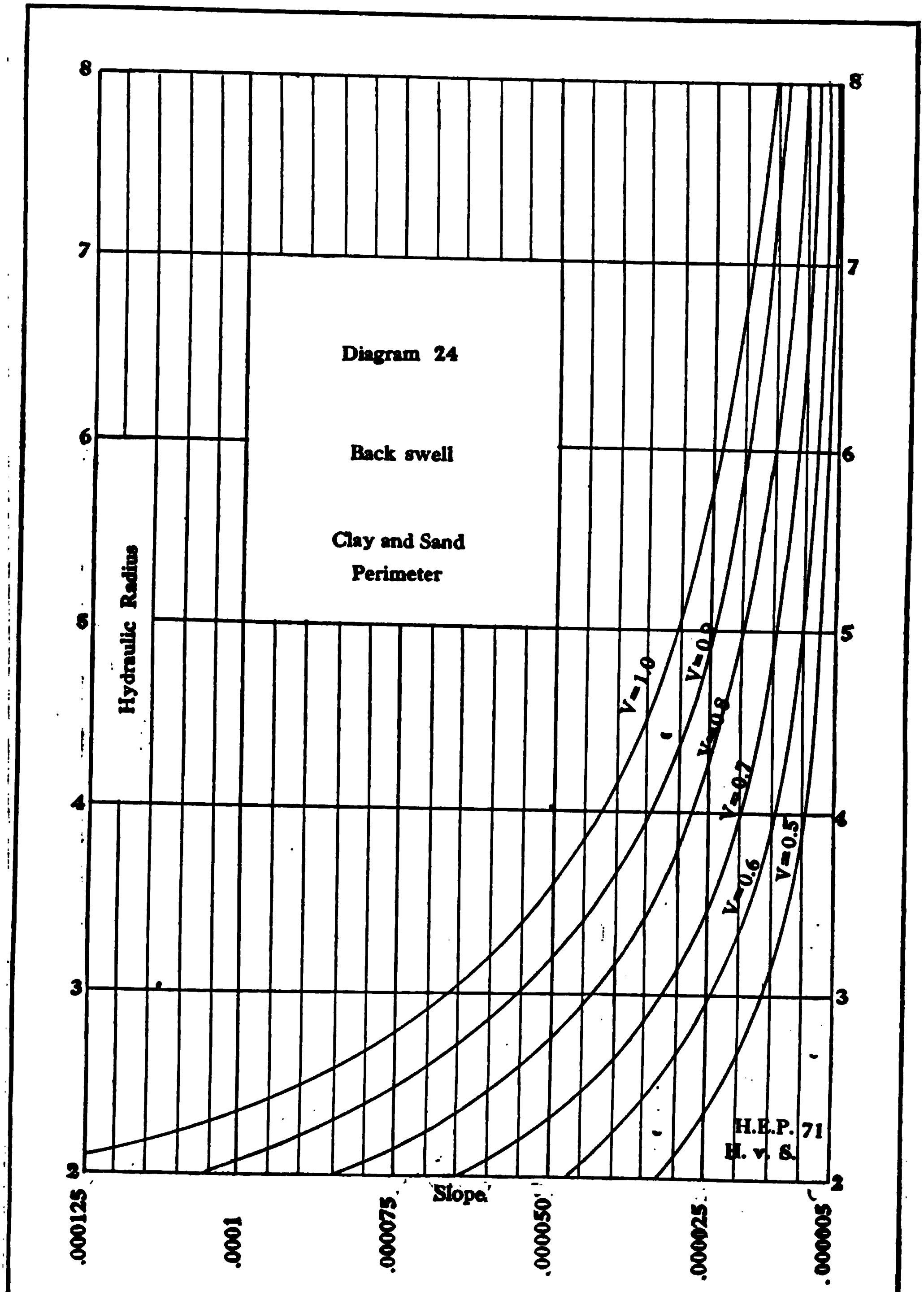


Diagram 24
Back swell
Clay and Sand
Perimeter
Hydraulic Radius
Slope
8
7
6
5
4
3
2
.000125
.0001
.000075
.000050
.000025
.000005
V=1.0
V=0.9
V=0.8
V=0.7
V=0.6
V=0.5
H.E.P. 71
H. v. S.

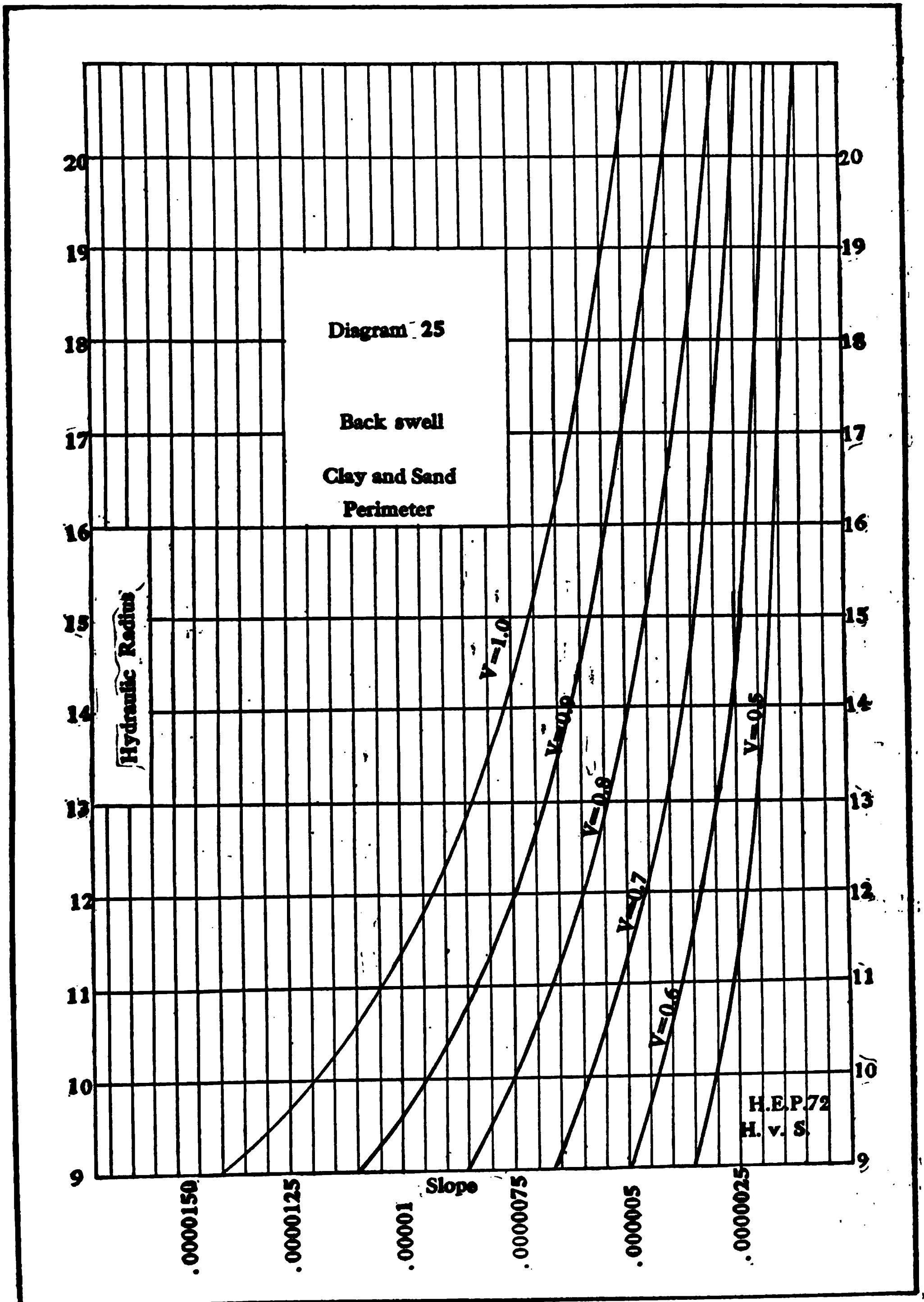


Diagram 25
Back swell
Clay and Sand
Perimeter
Hydraulic Radius
Slope
V=1.0
V=0.9
V=0.8
V=0.7
V=0.6
V=0.5
H.E.P.72
H. v. S.
20
19
18
17
16
15
14
13
12
11
10
9
.0000150
.0000125
.00001
.0000075
.000005
.0000025

tions continue, which means that the velocity will be much greater and therefore also S; for instance, where

$$P = 256, \quad A = 866, \quad R = 3.4, \quad \text{and} \quad V = 3.0, \quad S = 0.00085,$$

and thus the approximate head of the pond can be readily fixed.

In addition to determining the backswell for the volume of the normal flow, it must also be found for other river stages, especially for the high flow, in order that the full effect of ponding the water to a certain height at the dam may become clear.

If the flood flow is, for instance, in this case = 9600 c. sec. feet, the volume passing over the spillway is 9600–2600 = 7000 c. sec. feet, or 35 c. sec. ft. per linear foot of spillway crest, the overfall being

$$= 4.9 \text{ feet.}$$

At section A the volume of water will be increased approximately by a depth of five feet, and, assuming the banks to slope up uniformly,

$$P = 257, \quad A = 4500, \quad R = 17.9, \quad \text{and} \quad V = 2.13, \quad S = 0.000032,$$
$$\text{and the fall in first reach} = 0.160;$$

at section B, under like conditions and assumptions,

$$P = 165, \quad A = 3200, \quad R = 19.4, \quad \text{and} \quad V = 3.0, \quad S = 0.000073,$$
$$\text{and the fall in this second portion of pond} = 0.219;$$

at section C, again assuming uniform enlargement of area.

$$P = 258, \quad A = 3900, \quad R = 15.1, \quad \text{and} \quad V = 2.46, \quad S = 0.000067,$$
$$\text{and the fall in this reach} = 0.264:$$
$$\text{therefore the total fall in the 12,000 feet of pond} = 0.643$$
$$\text{as against 0.104 during the normal flow stage.}$$

Carrying this investigation to the head of the pond, assumed for normal stage, the flood rise at that location will become apparent, and this, compared with the known flood height at that point before the dam is erected, will show what increase will be caused by such a dam.

It is the author's experience that this subject of backswell receives generally but scanty attention, and often is altogether ignored, in con-

sequence of which the first flood brings in its wake numerous damage claims for inundations of lands, highways, and railroad tracks, while bridges and buildings may be endangered, all of which might have been avoided had a proper knowledge of the extent of the backswell been secured before the dam was constructed, as considerable control of it can be secured by its proper design, as will appear later in this discussion. At any rate, the consideration of the height of the spillway is incomplete unless the backswell is fully determined.

ARTICLE 58.—*The length of the spillway* is primarily that which is required to pass the maximum flood flow within the limit of a certain height of overfall over the spillway crest, and this height will be largely determined by the elevations of the natural banks or of the embankments to be erected and by the volume of the flood flow; aside from this consideration the spillway should be as short as possible. However, in alluvial formations it will generally be most advisable to make the spillway length equal to the width of the stream bed, as contracting this is always fraught with danger which will have to be counteracted by costly abutment and embankment structures.

The flood flow is found from a rating table of the river's discharge compiled from flow measurements (Art. 41) or from flow computations (Art. 44); from this the volume to be diverted for the power development is deducted, the residue is to be passed over the spillway. The maximum overfall height having been determined, the weir discharge for this height per foot length is found (Diagram 2), the total volume to be passed is divided by this, and the quotient represents the required spillway length.

Example.

Flood flow assumed at	=	9000 cub. sec. ft.
Diverted flow assumed at	=	1000 cub. sec. ft.
Volume to be passed	=	8000 cub. sec. ft.
Maximum overfall height	=	3.5 feet
Spillway discharge per foot of length	=	21.5 cub. sec. ft.
Spillway length required $\frac{8000}{21.5}$	=	372 feet.

As the spillway will be provided with waste flumes, by the aid of which the pond can be drawn down, their discharge capacity will be available for additional flood passage, and they therefore represent a safety factor in this respect.

ARTICLE 59.—*The amount of pressure and resistance* is the first subject to be considered when the spillway section is to be designed. The pressure P of a column of water restrained in its natural passage is represented by the product of the pressure area A′ and the weight of water W; the area factors are height of water column H and the length of pressed surface.

In Fig. 20 S is vertical, H = S, ordinates and abscissæ represent factors of pressure area A′ which intersect in pressure plane a b.

$$A' = S\ (\text{base}) \times \frac{H}{2}\ (\text{half altitude}) = \frac{H^2}{2}.$$

When the water level is below the top of the pressed surface, the pressure area decreases in like ratio, both S and H being reduced by h_1, or

$$H = S - h_1 \text{ and } A' = \tfrac{1}{2}(S - h)^2.$$

When the water stands above the pressed surface, the pressure area becomes a trapezoid.

In Fig. 21 S is vertical, S = H − h,

$$A' = H - h\ (\text{base}) \times \frac{H + h}{2}\ (\text{altitude})$$
$$= \frac{H^2 - h^2}{2};$$

or expressed in S

$$A' = S \times \frac{S + h + h}{2}$$
$$= \frac{S^2}{2} + S\,h.$$

The same result is illustrated geometrically in Fig. 22, where triangle b g l represents area due to H − h and S. Trapezoid b c f l is area when $H = S + h = \frac{S^2}{2} + S\,h$, by which the rectangle b c k l = S h which = parallelogram b c f g, is added to triangle b g l. Or for expression in values of H, $A' = \frac{H^2 - h^2}{2}$, being the trapezoid b c f l which consists of half the square a e f l = $\frac{H^2}{2}$ less half the square a b c d = $\frac{h^2}{2}$.

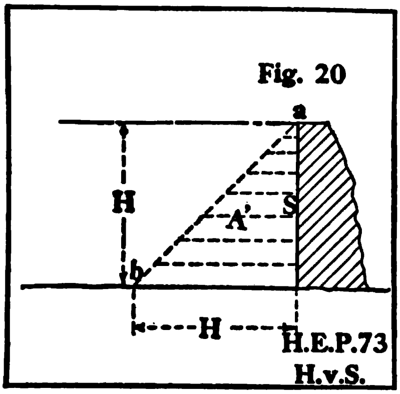


Fig. 20

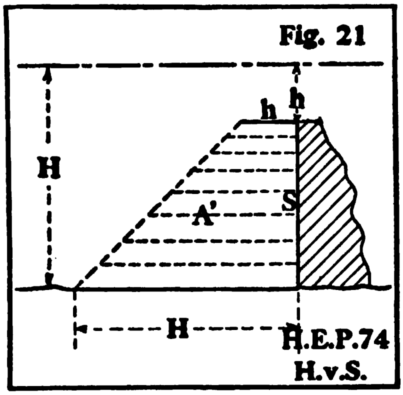


Fig. 21

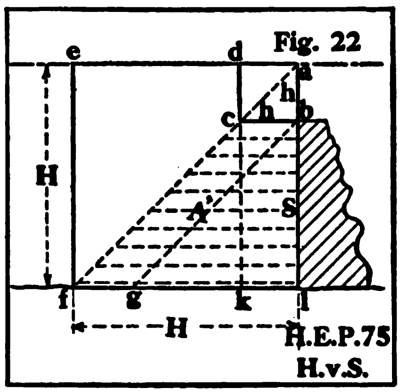


Fig. 22

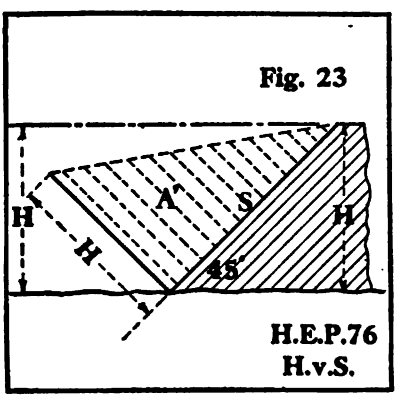


Fig. 23

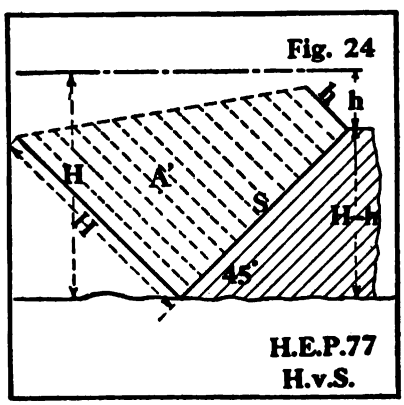


Fig. 24

The same pressure area expressions apply when the pressed surface is inclined, but the values vary with the inclination of S.

In Fig. 23 A′ is a triangle, S is base, H is altitude; $A' = \frac{S\,H}{2}$.

In Fig. 24 A′ is a trapezoid, S is base, $\frac{H+h}{2}$ is altitude; $A' = S \times \frac{H+h}{2}$.

The product of the pressure area and the weight of water (62.5 pounds per cubic foot) is the pressure "P" against the surface, which when the latter is vertical and

$H = S$	$P = 31.25\,H^2$	F. 1
$H = S - h$	$P = 31.25\,(S - h_1)^2$	F. 2
$H = S + h$	$P = 31.25\,(H^2 - h^2)$	F. 3

When S is inclined and

$H = \frac{S}{\sqrt{2}}$	$P = 31.25\,H\,S$	F. 4
$H = S - h_1\sqrt{2}$	$P = 31.25\,H\,(S - h_1\sqrt{2})$	F. 5
$H = \frac{S}{\sqrt{2}} + h$	$P = 31.25\,S\,(H + h)$	F. 6

The intensity of the pressure of any volume is concentrated in its gravity plane passing at right angles to the pressed surface through its *centre of gravity G.*

In a triangle lines drawn from apex to bisect opposite sides are gravity lines, and the centre of gravity lies at their intersection and one-third of the altitude above the base.

In a trapezoid the centre of gravity is found as in Fig. 25 by extending the base lines in opposite directions to equal their sum, connecting the ends of these extensions, d i, and connecting bisects of the base lines as b f; the intersection of d i and b f marks the centre of gravity.

The height of the centre of gravity above the base g e =

$$\frac{2\,a\,c + g\,e}{a\,c + g\,e} \times \frac{h'}{3} \text{ (h′ = altitude of trapezoid).}$$

When the pressure acts upon a body, its force is expressed by the product of its intensity and the *lever arm* through which this pressure is applied.

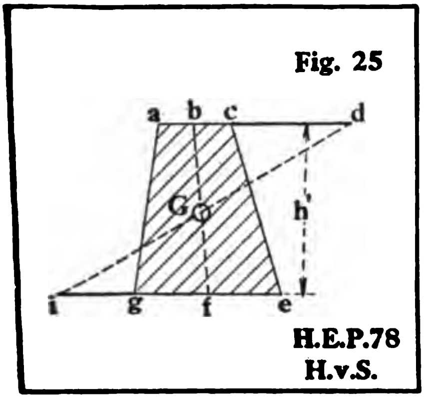


Fig. 25

H.E.P.78
H.v.S.

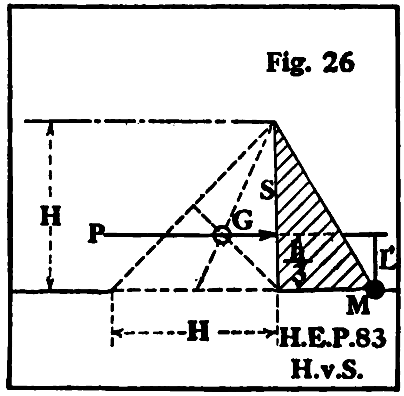


Fig. 26

H.E.P.83
H.v.S.

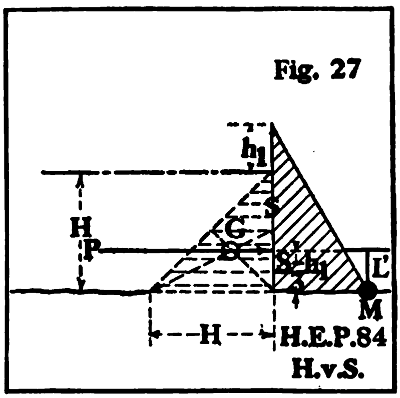


Fig. 27

H.E.P.84
H.v.S.

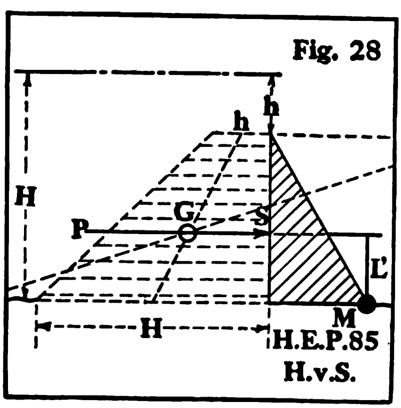


Fig. 28

H.E.P.85
H.v.S.

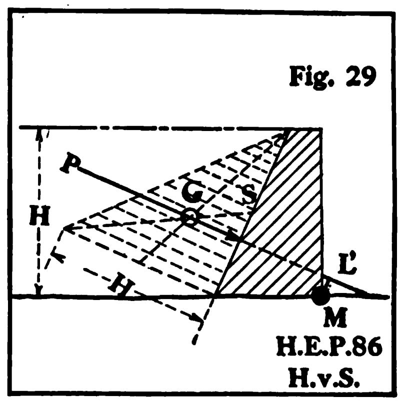


Fig. 29

H.E.P.86
H.v.S.

The lever arm L is the vertical distance from the pressure line to the turning point M of the pressed body.

In Fig. 26 S is vertical, A′ is triangle, S = H, G = $\frac{H}{3}$, and, as the pressure passes through G in a direction vertical to S, L′ = $\frac{H}{3}$, and the dynamic force M P = $\frac{P\,H}{3}$, and of water M P = 10.417, H^3 . . . F. 7

In Fig. 27 S is vertical, H = S − h_1, L′ = $\frac{S - h_1}{3}$,

$$M\,P = 10.417\,(S - h_1)^3 \ldots\ldots F.\ 8$$

In Fig. 28 S is vertical, H = S + h, L′ = $\frac{H + 2\,h}{H + h} \times \frac{S}{3}$

$$M\,P = 10.417\,(H^3 - 3\,H\,h^2 + 2\,h^3) \ldots\ldots F.\ 9$$

Diagrams 26 to 29 give M P for different values of H and h.

In Fig. 29 S is inclined; the pressure acts vertical to S at M through the lever arm L′, the value of which varies with inclination of S.

In Fig. 30 P intersects M and L, therefore becomes zero, and as S becomes more inclined P intersects the base of the body. For the practical solution of the dynamic forces acting on inclined surfaces the pressure is analyzed into its horizontal and vertical components, h P and v P.

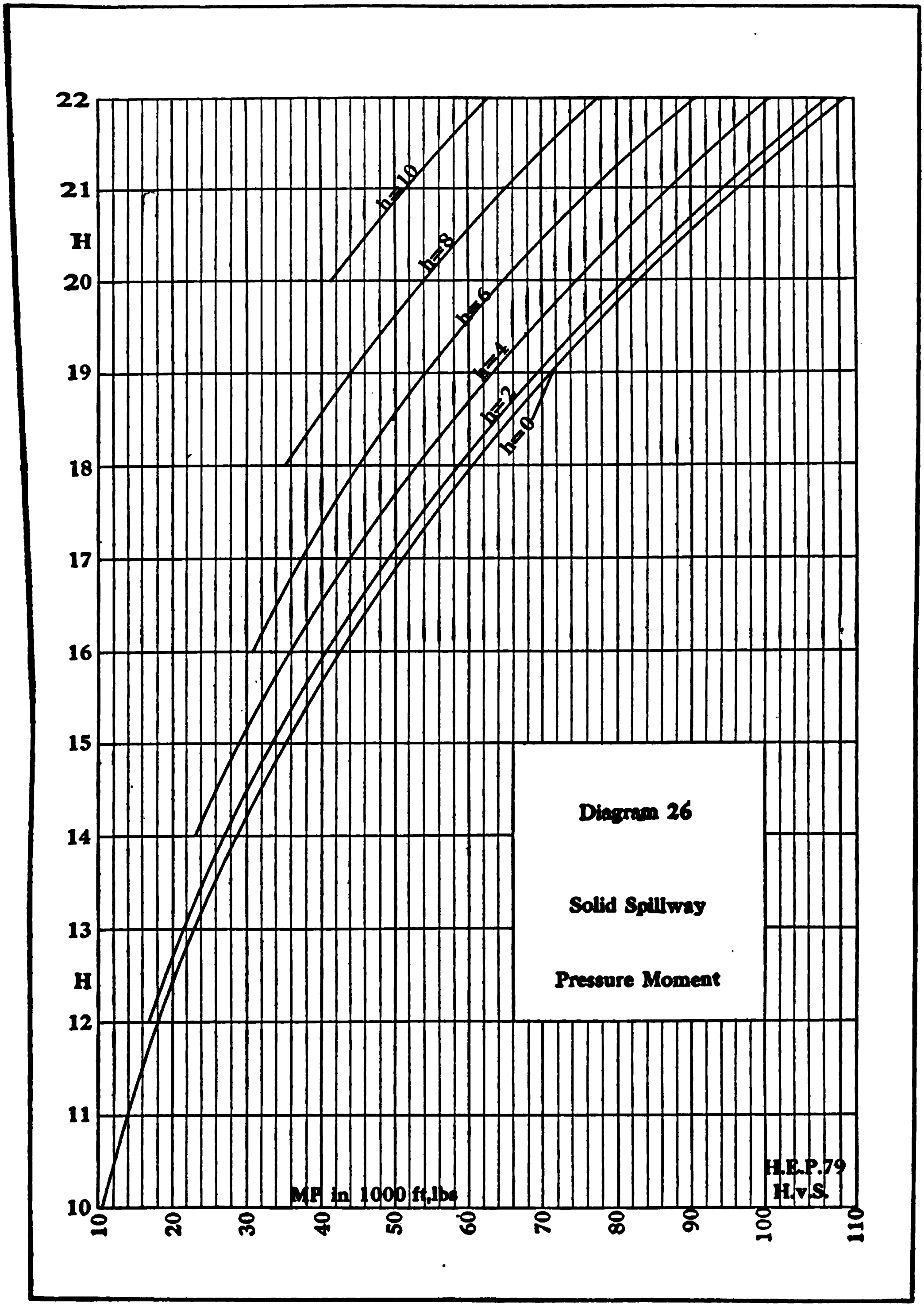
In Fig. 31 S is inclined 45° and = H $\sqrt{2}$, P is the total pressure acting perpendicular to S; a b c d is a square of which P is the diagonal, and h P = $\frac{P}{\sqrt{2}}$, S = H $\sqrt{2}$, from F. 4, P = 31.25 S H and h P = $\frac{31.25\,S\,H}{\sqrt{2}}$. Inserting value of S, = $\frac{31.25\,H^2\sqrt{2}}{\sqrt{2}}$ = 31.25 H^2, which is the same as P in F. 1, when S is vertical and = H.

The horizontal component of P equals P in Fs. 1, 2, and 3 as per height of water, no matter what the inclination of S is. The vertical component of P is the weight of the water area overlying the base e f.

M P for inclined surfaces is the product of h P and L, and therefore the same as expressed in Fs. 7, 8, and 9, to wit: when

$H = \frac{S}{\sqrt{2}}$	$M\,P = 10.417\,H^3$	F. 7
$H = S - h_1\sqrt{2}$	$M\,P = 10.417\,(S - h_1)^3$	F. 8
$H = \frac{S}{\sqrt{2}} + h$	$M\,P = 10.417\,(H^3 - 3\,H\,h^2 + 2\,h^3)$	F. 9

When a structure is to restrain water, P and M P must not only be counteracted but resisted by greater forces, otherwise the effect of these


22
21
H
20
19
18
17
16
15
14
13
H
12
11
10
10
20
30
40
50
60
70
80
90
100
110
h=10
h=8
h=6
h=4
h=2
h=0
Diagram 26
Solid Spillway
Pressure Moment
MP in 1000 ft.lbs
H.E.P.79
H.v.S.

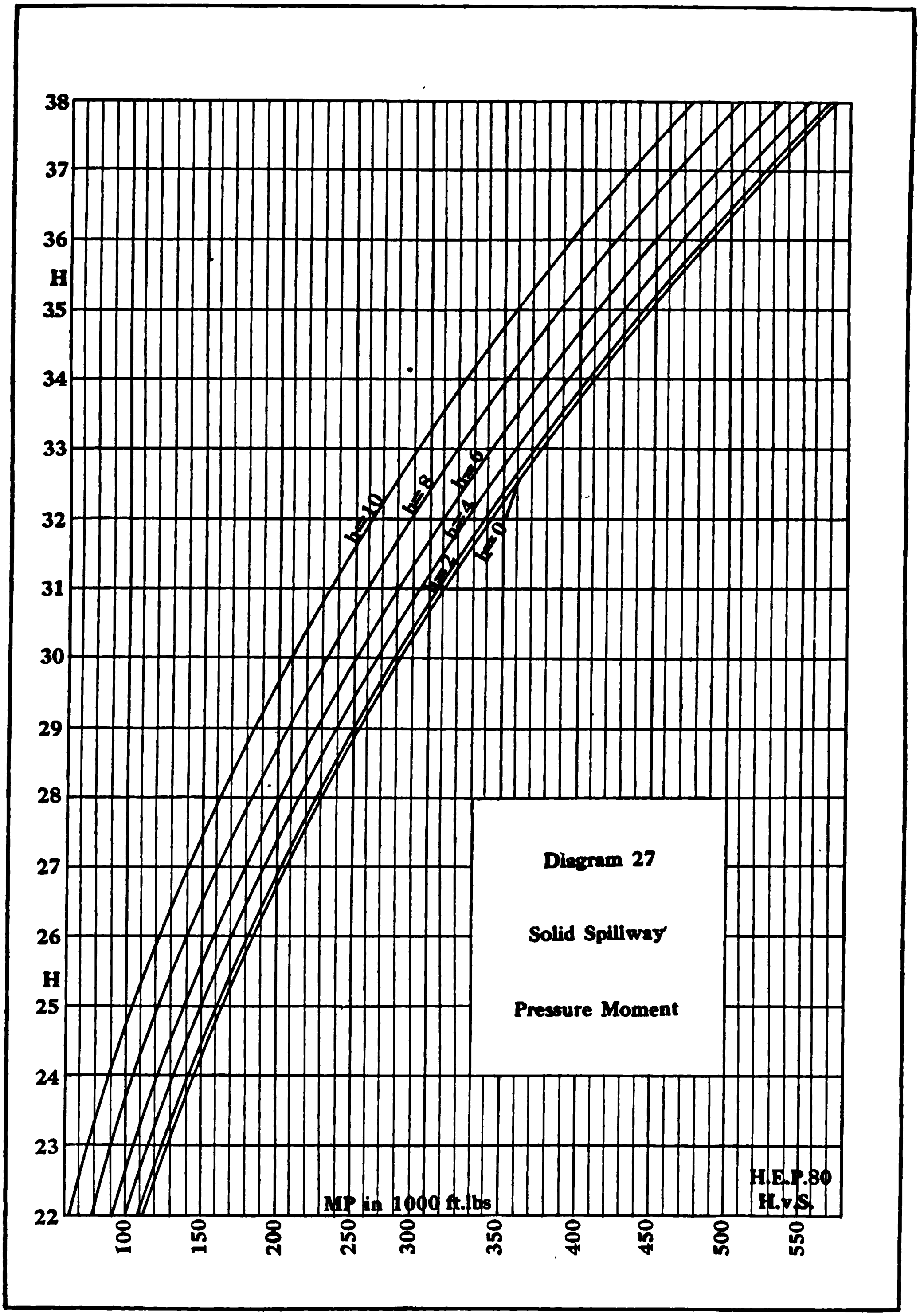
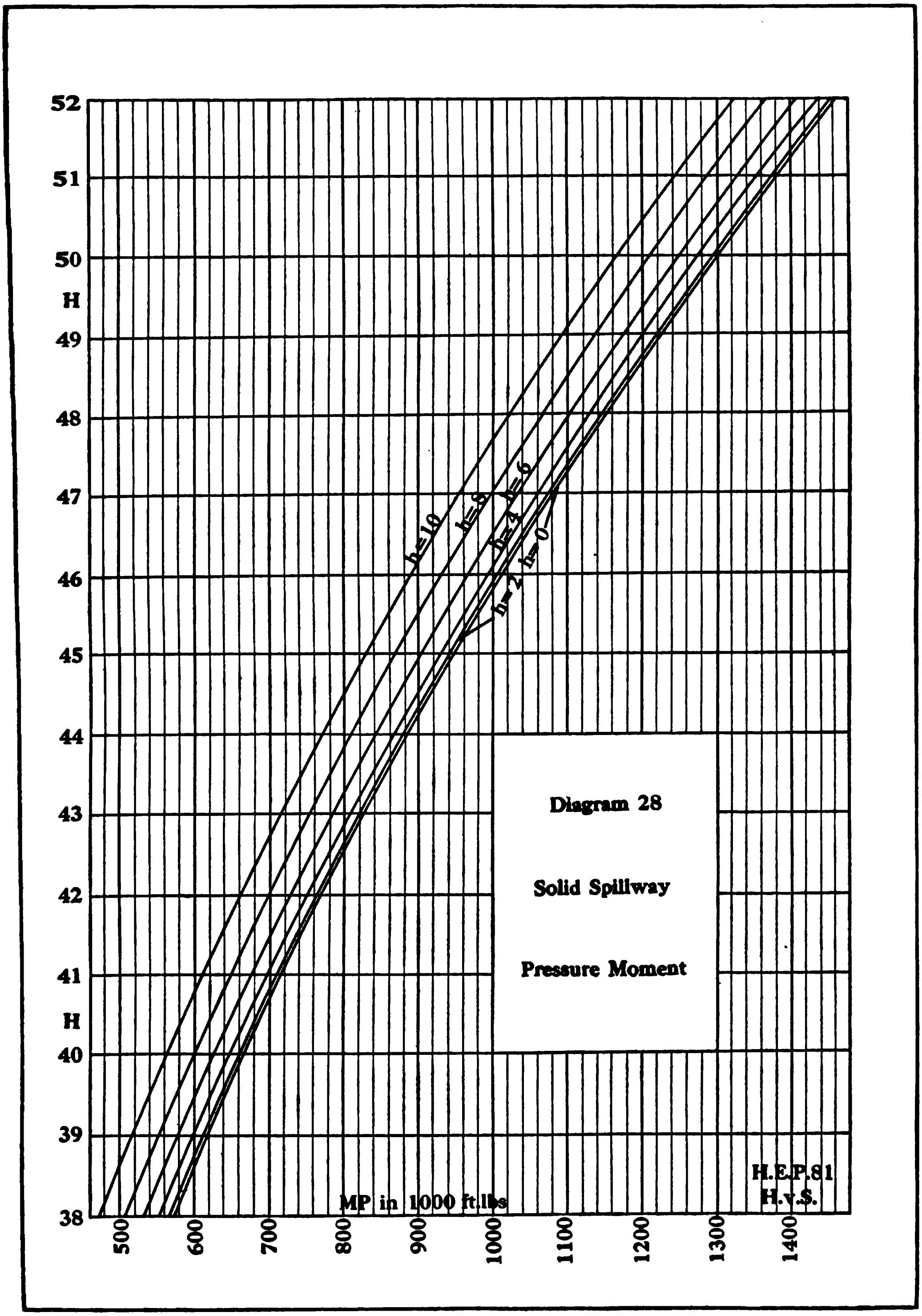
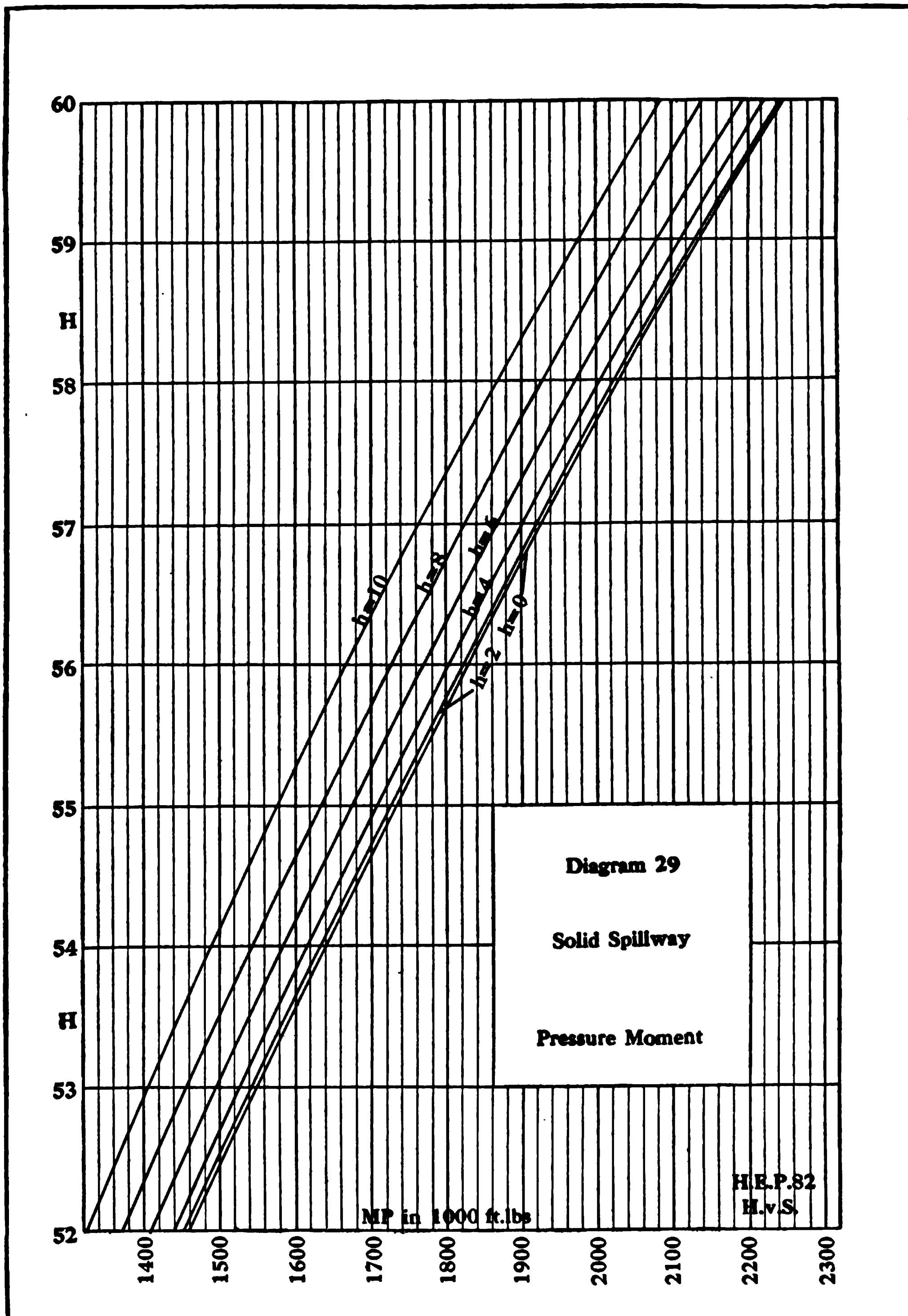


Diagram 27
Solid Spillway
Pressure Moment
H
38
37
36
35
34
33
32
31
30
29
28
27
26
25
24
23
22
MP in 1000 ft.lbs
100
150
200
250
300
350
400
450
500
550
h=10
h=8
h=6
h=4
h=2
h=0
H.E.P.80
H.v.S.

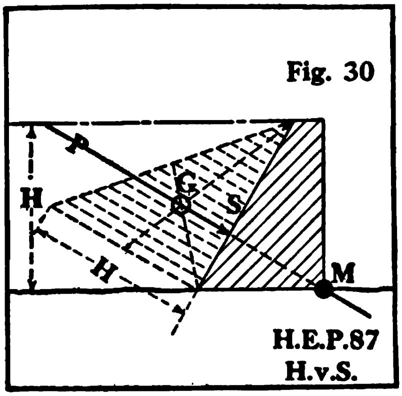


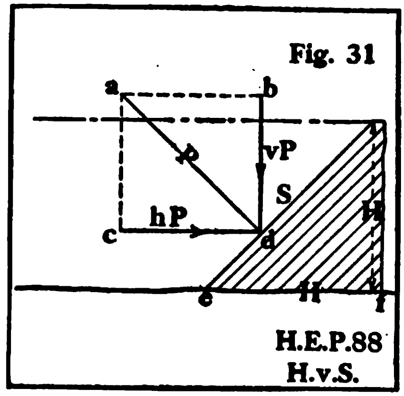
52
51
50
H
49
48
47
46
45
44
43
42
41
H
40
39
38
h=10
h=8
h=6
h=4
h=2
h=0
Diagram 28
Solid Spillway
Pressure Moment
H.E.P.81
H.v.S.
MP in 1000 ft.lbs
500
600
700
800
900
1000
1100
1200
1300
1400


60
59
H
58
57
56
55
54
H
53
52
h=10
h=8
h=6
h=4
h=0
h=2
Diagram 29
Solid Spillway
Pressure Moment
H.E.P.82
H.v.S.
MP in 1000 ft.lbs
1400
1500
1600
1700
1800
1900
2000
2100
2200
2300

will be to displace the structure by sliding it along the base or any horizontal plane, by overturning it around its toe, crushing its material, or rupturing the structure.

ARTICLE 60.—*Sliding* would be caused by h P, which must be met by the effective weight W of the resisting body, being the product of its material weight, plus the vertical water pressure v P on its horizontal projection, and the friction coefficient "f" developed at the horizontal joints and at the base between the structure and the material on which it rests. When the structure is a homogeneous, impervious mass, its material weight is the product of its area A and the unit weight W of the substance composing it, which is not diminished by standing in water; if water





enters under its base and remains there confined, an upward pressure is created which is equal to the product of the area of the base, the depth of its centre of gravity, and 62.5 pounds; if it freely escapes from under the base, no such upward pressure exists. The friction coefficient f is the proportion of ultimate friction between two masses to the perpendicular pressure,—*i.e.*, the weight of the upper. Ex.: if it requires a force of 600 pounds to move a 1000 pound stone resting upon a horizontal surface, $f = \frac{600}{1000} = 0.60$; when surfaces are connected in any manner, resistance to sliding is represented by the cohesion of the bond or connection which is added to the frictional resistance.

In the designs herein to be considered for spillways the structures are assumed to consist of a homogeneous, impervious mass; water is prevented from entering beneath the base; the foundation is connected to the underlying material and the superstructure is bonded to the foundation.

Resistance to sliding is theoretically found in the weight of the superstructure plus the vertical water pressure; but in practice it is recommendable to ignore the latter, excepting in those designs which are specifically based upon gravity theory, that is when the pressed surface is inclined from the vertical. As a general rule, therefore, the weight of the structure is to equal the horizontal component of the water pressure,

$$Aw = \text{or} > h\ P.$$

ARTICLE 61.—*Overturning* of structure is caused by the dynamic force of the horizontal component of the water pressure, h P L′, and is theoretically resisted by the moment of weight, M W, being the sum of the products of weight of structure into its lever arm, A w L, and of the vertical water pressure into its lever arm, v P L″, or

$$M\ W = A\ w\ L + v\ P\ L'',$$

provided the resultant of these two forces cuts the base of the structure.

In Fig. 32 G is the centre of gravity of the structure, M its fulcrum or toe, O is the locus of the gravity plane in the base of the structure, O M = L the lever arm through which W acts.

In Fig. 33 G O = L′ is the lever arm of P or h P, O M = L is the lever arm of A w, N M = L″ is the lever arm of v P, G P = P or h P, G W = A w or A w + v P, G M′ = R, the resultant of h P and G W, O′ = the locus of the resultant in the base of the structure.

When the pressed surface is inclined, v P is credited to W as adding to the resistance against overturning, the structure being of a gravity design.

The resultant must always fall into the middle third of the base, for reasons given in next article.

Resisting sliding and overturning guarantees stability of position by A w L = or > h P L′ and O′ falling in middle third of the base.

ARTICLE 62.—*The crushing or rupturing* of the structure may be caused by the concentration of excessive pressures on parts of it.

In Fig. 34 the weight W of the rectangular structure c d e f acts in the gravity plane on the centre of the base B and is transmitted to the material on which the structure rests. The pressure due to W is represented by the reaction rectangle e f i j = W, in which the ordinates P, P′, etc., represent the pressure and its distribution in magnitude as per length of ordinates. If the material beneath the structure does not yield

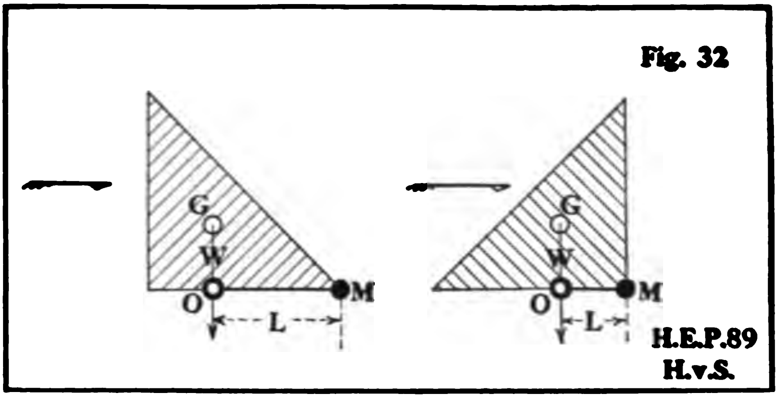


Fig. 32

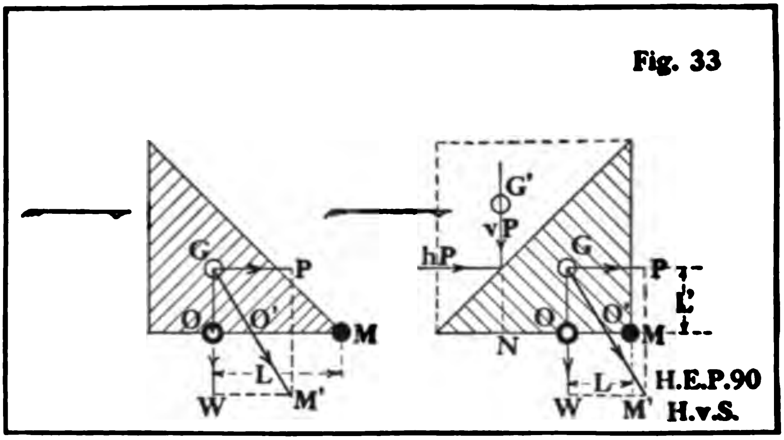


Fig. 33

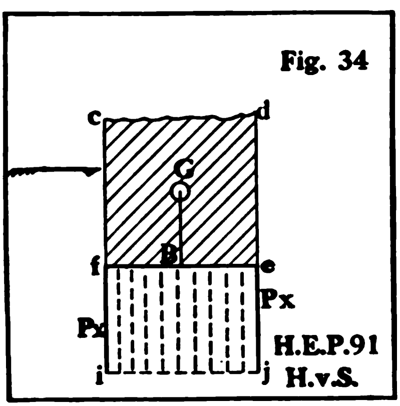


Fig. 34

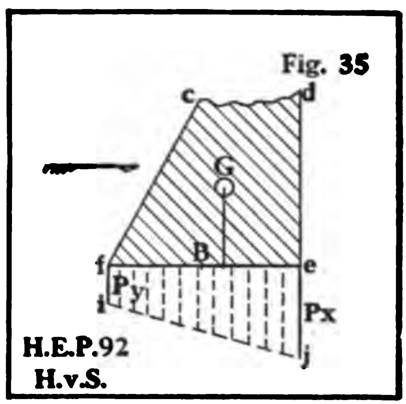


Fig. 35

to these pressures, they react on the structure, and the resistance therein developed must be sufficient to safeguard the material against crushing.

Rigidity of foundation is herein presupposed, and therefore the pressures tend to crush the structure's mass.

In Fig. 35 the structure is a trapezoid c d e f, W acts on the base B at some point near $e = \frac{B}{2} - b$; the pressure is represented by the reaction trapezoid e f i j,

$$\text{or } \frac{Px + Py}{2} B = W.$$

The pressure is not uniformly distributed over the base, but is maximum at the end nearest to the gravity plane and minimum at the other, its ratio between these being proportional. The mean pressure ordinate lies in the gravity plane produced, because the sums of pressures on both sides of this plane are equal as represented by reaction diagram; the distribution of the pressures therefore depends upon the locus of the gravity plane in the base B and essentially on its distance from the centre of B.

Fig. 36 represents the reaction trapezoid e f i j, the locus of the gravity plane is at O and its distance from the centre of the base B = b, which is determinable from the formula for distance of centre of gravity above the base of a trapezoid, to wit:

$$e\,O = y = \frac{P + 2\,P'}{P + P'} \times \frac{B}{3} \text{ (P and P}' = \text{Px and Py);}$$

substituting for P′ its expression from

$$\frac{P + P'}{2} \times B = W,$$

$$y = \left[(P + \frac{4\,W}{B} - 2\,P) \div (P + \frac{2\,W}{B} - P)\right] \frac{B}{3};$$

solving for P,

$$P = \frac{4\,W}{B} - \frac{6\,W\,y}{B^2}, \text{ inserting } y = \frac{B}{2} - b,$$

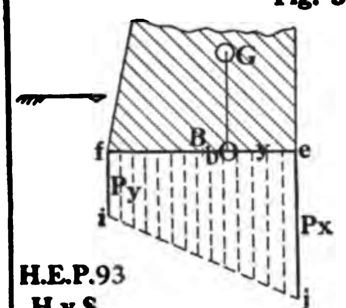
$$P = \frac{W}{B} + \frac{6\,W\,b}{B^2}, \text{ assigning to "b" a value of } 0.1\ B,$$

$$P = \frac{1.6\ W}{B}, \text{ and placing this in } \frac{P + P'}{2}\,B = W\,,$$

$$P' = \frac{0.4\ W}{B}, \text{ which are the expressions for the maximum and}$$

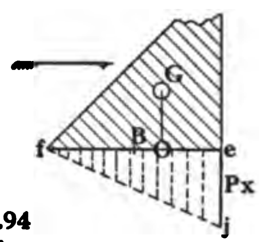
minimum pressure ordinates in the reaction trapezoid on the base of structure or on any horizontal plane of the structure, W representing the superposed weight. When b is a function of the locus of resultant of total pressures against the structure, P, P′, etc., are expressions of total pressures due to structure's weight and of pressure moment of water column restrained by the structure.

Fig. 36



H.E.P.93
H.v.S.

Fig. 37



H.E.P.94
H.v.S.

From Fig. 37 it is evident that the distance between the locus of resultant and the centre of the base, "b", controls the distribution of the pressure.

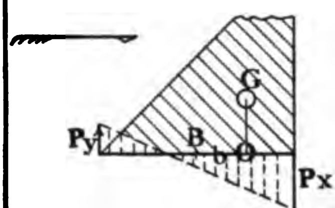
$$\text{When b is} = \frac{B}{6},\ P = \frac{2\ W}{B} \text{ and } P' = \text{zero},$$

the pressure distribution is then represented by the triangle e f j and the maximum, P, is double the mean pressure. This condition represents the limit within which the pressure strains are met by compression in the structure; any further moving of O from the centre of the base, or any increase of b beyond $\frac{B}{6}$, creates tension in the superstructure.

In Fig. 38 b exceeds $\frac{B}{6}$, the reaction forces are represented by a positive and negative triangle, the latter represents tension developed at the base of the structure, by which the pressure theories heretofore discussed undergo a complete change.

This condition prevails, therefore, whenever b exceeds $\frac{B}{6}$, and the structure may then be ruptured in the portion where tension stresses are developed. Safety against crushing must therefore be secured by so proportioning the section that the maximum pressure P is within the limits of the strength of the material, while the pressure resultant R must fall in the middle third of any horizontal joint and of the base.

Fig. 38



H.E.P.95
H.v.S.

Upon these theories the spillway design must be based, the desideratum being the most economical section in area which meets the requirements, guaranteeing stability of position and safety of structure, which practically resolves itself into obtaining a value for M W which exceeds M P by some margin called the safety factor, S f.

Article 63.—What a *safety factor* should be must be decided for each case from those conditions which are likely to become important factors in developing the resistance capacity of the structure and those which may tax it.

The character of the foundation is one of these important elements, no matter what the spillway section is. It will be appreciated from the presentation of the stability theories that, with a foundation lacking in rigidity, several of the important theoretical deductions become unreliable in fact, as stated; preventing water from entering beneath structure and placing it on an unyielding mass are essential in theorizing from causes to effects.

When set upon a hard rock ledge and securely keyed into it, the structure may be regarded as having become a part of it, and it will develop its resistances to the fullest capacity. This cannot be expected in alluvial locations; water penetration can be guarded against efficiently

by proper cut-off construction, and rigidity may be obtained by a correctly planned pile-bearing foundation; but, after all, the responsibility is merely transferred to the substance in which the piles stand. In one case in the author's experience the underlying material, to the depth of fifty feet, consisting of clay and sand, bodily moved several inches downstream; these conditions, therefore, when compared with rock location, call for fundamental increase of obtainable safety.

The abutments may enhance the spillway's resistance; when these are natural rock banks, the structure, unless of considerable length, gains from them greater security than from abutments constructed in or against alluvial banks, not merely because there is less possibility of water finding a way around them, which should be absolutely guarded against, but because of the rigidity afforded by the natural bulwarks.

The height of overflow is an important condition; though fully credited in the respective design in accord with sound theory, it is nevertheless obvious that the larger the natural forces the greater the possibilities of failures. In general high overfalls are best avoided, but, where they must be reckoned with, the safety factor should be prudently adapted.

Prevalence of trees and logs passing over spillway must be met by an increase in section over that normally required. In northern latitudes heavy ice is likely to form and subject the structure to abnormal thrust which finds no proper resistance factor in the theoretical design, or ice may gorge upstream of the spillway during the spring break-up, as occurred during the last winter in the Alleghany and Monongahela rivers and brought destruction to several dams.

And finally the height of the spillway itself has a most important place in this category. High dams have been constructed a century or more; not so, however, with spillways; designing a structure a hundred feet or higher over which large volumes of water are pouring with consequential shocks of the leap of a Niagara is quite a problem apart from that of a dam of similar height restraining a pond of quiet water. It would be a serious fallacy to lay down a rule of designing spillways of any and all heights with a fixed safety factor of two or whatever measure.

It is a good enough business axiom to apportion the insurance to the probable loss, and this has its place here; when the failure of a spillway is likely to work great destruction to property and, perhaps, of human life, the possibility of such a happening should be considered sufficient reason for an increase of the safety factor.

The following is suggested as a basis for *determination of the safety factor:*

1. For spillways founded on rock ledge with natural rock abutments .... S f = 2.00
2. For spillways founded on rock ledge with constructed abutments......... S f = 2.25
3. For spillways founded on alluvial material with constructed abutments ... S f = 2.50
4. For overfall in excess of 0.20 of spillway height add per foot of such excess to above 0.10
5. For each five feet of height of spillway in excess of 50 feet add to above 0.10
6. When failure would cause great destruction add to above...................... 0.25

Example.—For a spillway 40 feet high, founded on alluvial material, with maximum overfall of 10 feet, and not in close proximity (5 miles) to any settlement, the safety factor should be taken at $2.5 + 0.2 = 2.70$, or a spillway 65 feet high, on rock bed, with constructed abutments and a maximum overfall of 8 feet, should be designed with a safety factor of $2.25 + 0.30 = 2.55$.

ARTICLE 64.—Having determined the height of the spillway, the maximum overfall, and the safety factor, *the theoretical design* is worked out, and by finding the smallest section in area in which the resultant falls in the middle third of the base and M W = M P × S f, the other stability requirements will be satisfied until certain limits of height are reached, as will appear in the further discussion.

Fig. 39 represents sections of an equilateral triangle, the perpendicular being the upstream or pressed surface: condition 1, without water pressure,—*i.e.*, when the pond above the spillway is drawn down; condition 2, when the water stands at the top, representative of the low stage, all the flow being diverted to the power station; condition 3, with normal overflow; and condition 4, with highest overflow under which the stability requirements are satisfied. The sections are 15 feet high; their material weight is taken at 140 pounds per cubic foot; the areas contain 112.5 square feet; the weights are expressed in short tons, the pressure ordinates in pounds.

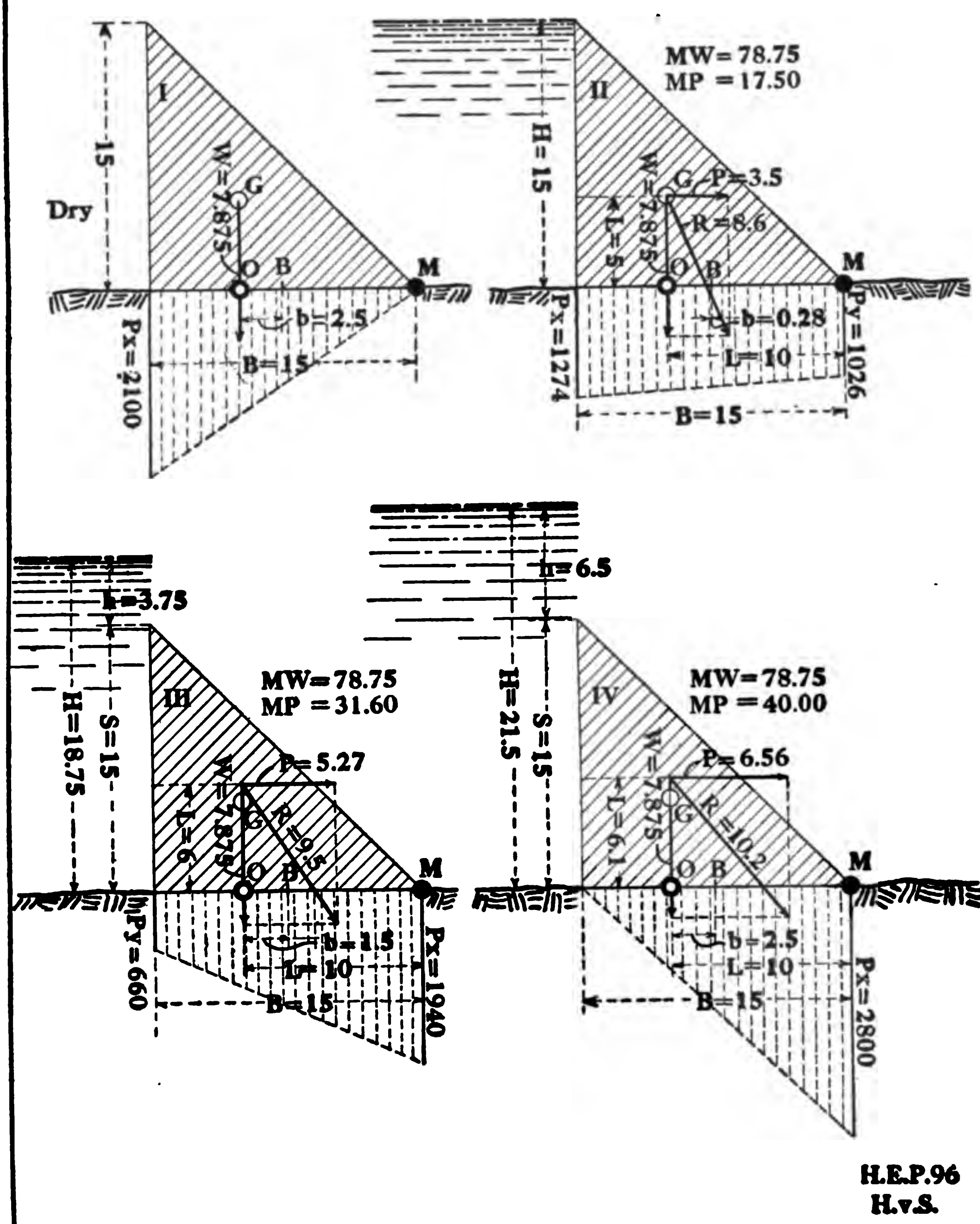
Sec. I. No water pressure, $W = 7.875$, $b = 2.5 = \frac{B}{6}$, max. pressure on the base = 2100 lbs. at the upstream end.

Sec. II. Water at the top of the section, M W = 78.75, M P = 17.5, b = 0.28; the max. pressure on the base = 1274 lbs.

Sec. III. Water stands 3.75 feet over the section, M W = 78.75, M P = 31.6, b = 1.5, the max. pressure on the base = 1940 lbs.

Sec. IV. The water stands 6.5 feet over the section, M W = 78.75, M P = 40.0, $b = 2.5 = \frac{B}{6}$, max. pressure on the base = 2800 lbs.

Fig. 39


MW = 78.75
MP = 17.50
MW = 78.75
MP = 31.60
MW = 78.75
MP = 40.00
Dry
Px = 2100
Px = 1274
Py = 1026
Py = 660
Px = 1940
Px = 2800
B = 15
H = 15
H = 18.75
H = 21.5
S = 15
h = 3.75
h = 6.5
W = 7.875
P = 3.5
R = 8.6
P = 5.27
R = 9.5
P = 6.56
R = 10.2
b = 2.5
b = 0.28
b = 1.5
L = 10
L = 5
L = 6
L = 6.1
H.E.P.96
H.v.S.

These are representative of the fluctuating forces to which a spillway structure is exposed, from the dry pond to the maximum overflow condition, and the reaction diagrams illustrate the shifting of the pressures on the base from the upstream to the downstream end, being a forcible reminder of the necessity of rigidity in the foundation and correct analysis of the pressures for the overflow conditions; any rise above 6.5 feet will develop tension at the upstream end of the base and the structure will become unsafe.

Fig. 40 presents the same triangular sections as in Fig. 39, but in this case the hypothenuse is the pressed surface.

Sec. V. No water pressure, W = 7.875, max. pressure = 2100;
Sec. VI. The water stands at the top of the section

P = 5, h P and v P = 3.5, W is composed of

A W = 7.875 + v P = 11.375

M W is made up of A W L′ = 39.375 + v P L″ = 74.375

M P is made up of H P L′ = 3.5 × 5 as in sec. II = 17.5

R′ resultant of P and W = 5 and 7.875 = 12

$$b = 2.5 = \frac{B}{6} \text{ and max. pressure} = 1.6 \text{ tons.}$$

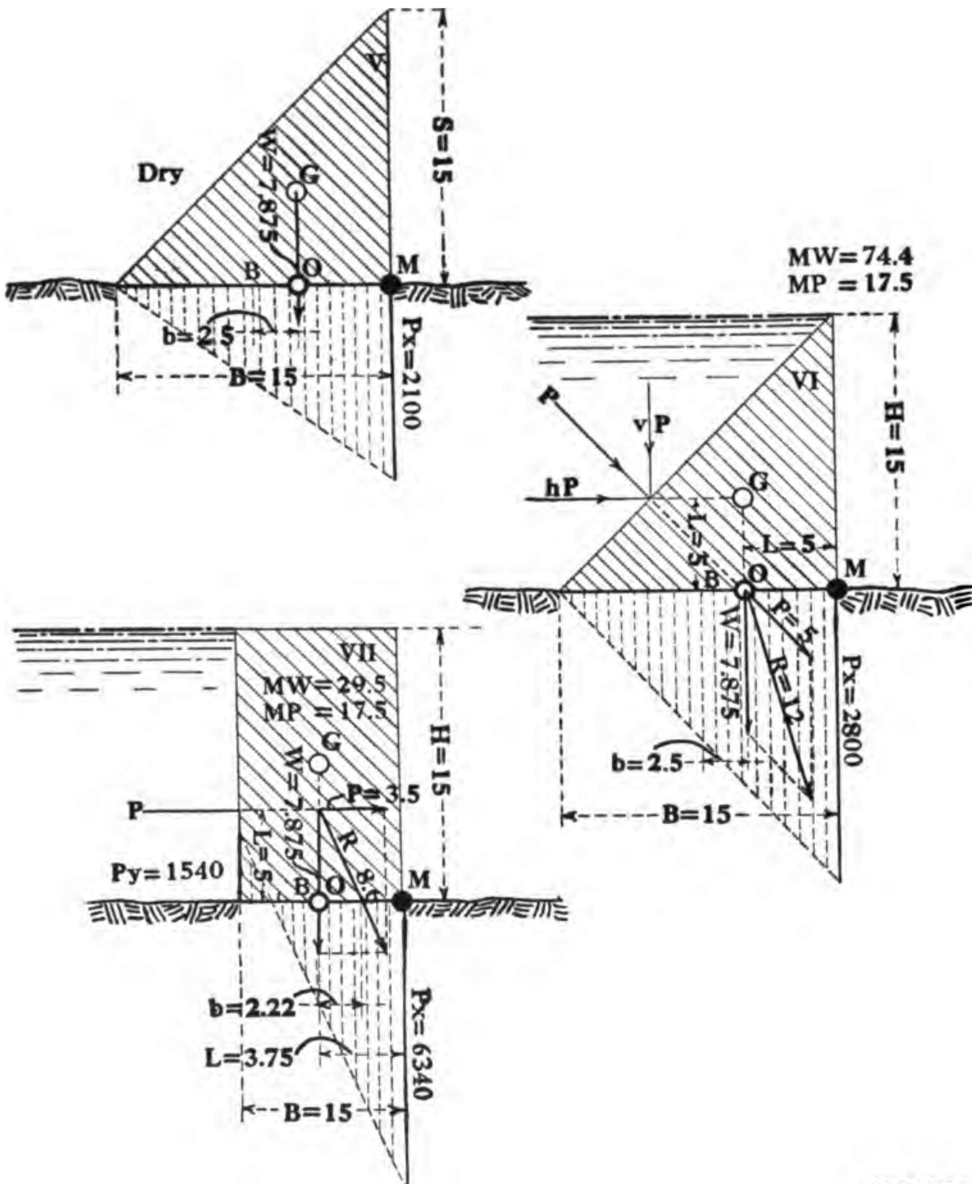
From this it is evident that this section will not meet the requirements that R′ fall in the middle third when water stands above it, as tension will be developed at the base and, though stability of position is amply safeguarded by a large safety factor, the structure will be endangered by reason of the tension stresses developed at the upstream end of base. Note that v P does not enter R′, which represents the total pressure resultant, but that it is a component of M W, as M P contains h P only; also that the pressure on the upstream end of the base is zero, with and without water pressure, because of the influence of v P in the latter case. The maximum pressure is, of course, increased under water pressure.

Sec. VII, Fig. 40, is a rectangle of the same area as the previous sections; its height is 15 ft., the base 7.5 ft.

Water stands at its top, M W = 29.5, M P = 17.5, b = 2.22 and the max. pressure 3.17 tons with negative pressure of 0.77 ton at the upstream end.

This section is apparently unstable for any water pressure condition.

Fig. 40



H.E.P.97
H.v.S.

ARTICLE 65.—*The practical design of a solid spillway* must conform to other conditions aside from those of stability considerations.

The top of the structure will be exposed to shocks from waves, logs, and ice, and must be given commensurate thickness; the overflowing water will plunge vertically on the downstream face of the spillway and on the river bed below unless that side is so formed that the volume of maximum overflow follows the spillway face and is guided in a direction by which the river bed will be protected against the water's force. The shocks against the spillway top will be most frequent when the overflow is shallow, then logs and ice hit the structure before clearing it; under these conditions, however, the velocity of the approaching water is low and these shocks are comparatively light. When the river is in flood, the overflow depth is correspondingly great and most of the floatage then clears the crest without touching it; the velocity of the water is, however, high, and if floatage then strikes the top the shock would represent great force. This would be the case when bridges, boat landings, and buildings are carried away, or large trees are precipitated with the caving banks into the stream; or when log and ice jams break loose and are hurled in large masses against the structure. The force of such shocks cannot be estimated, nor can the probability of such occurrences be ignored, and it is proper to take account of these possibilities when designing the spillway.

In practice reservoir dams are given a top width of one-tenth of their height, and that of spillways, generally speaking, should be twice this,—*i.e.*, the top width of the spillway should be two-tenths of the height of the spillway.

The investigation is now confined to the designing of the minimum area section in which

1. The pressed surface is vertical;
2. The top width equals two-tenths of the section height;
3. The downstream face is inclined to receive and guide the overfall;
4. The safety factor against overturning, with maximum overflow, exceeds two;
5. The locus of the resultant of pressures falls into the middle third of the section base; and
6. The maximum pressures do not exceed the safe strength of the material, which will be taken at ten tons per square foot.

A trapezoid represents this section, which will hereafter be referred to as "*the normal solid spillway section,*" and its proportions of design are:

Crest width equals two-tenths of height;
Base length equals eight-tenths of height;
Upstream face is vertical;
Downstream side is inclined one vertical in 0.6 horizontal;
Structure is of cyclopean, monolithic, or block concrete.

Fig. 41.—Spillway height....................S = 15 feet
Crest width.......................C = 3 feet
Base length.......................B = 12 feet
Spillway area, section..............A = 112.5 square feet.

In sec. I the upper pool is dry; stability against crushing is the only requirement to be met. The pressures are represented by the weight of the structure, which is taken at 140 pounds per cubic foot;

$$A\,W = 112.5 \times 140 = 15{,}750 \text{ lbs.}$$

The locus of the pressure line in the base, b, is found from the height of the horizontal gravity plane

$$G\,C = \frac{B + 2\,C}{B + C} \times \frac{S}{3} = 0.4\,S = 6 \text{ feet,}$$

and the length of the horizontal gravity plane

$$g\,p = \frac{(B - C) \times (S - G\,C)}{S} + C = 0.56\,S = 8.4 \text{ feet;}$$

then

$$b = \frac{B - g\,p}{2} - \frac{B}{2} = \frac{B - g\,p}{2} = 0.12\,S = 1.8 \text{ feet.}$$

Maximum pressure

$$P\,x = \frac{R}{B} + \frac{6\,R\,b}{B^2} = \frac{15750}{12} + \frac{170100}{144} = 2493 \text{ lbs.}$$

In sec. II, Fig. 41, the water stands level with the spillway crest; all characteristics are as in sec. I, and H=15 feet.

Water pressure	$P = 31.25\ H^2$	=	7031 lbs.
Its lever arm	$L' = \frac{S}{3}$	=	5 ft.
Pressure moment	$M\,P = 7031 \times 5$	=	35,155 ft. lbs.
Spillway weight	$W = 112.5 \times 140$	=	15,750 lbs.
Its lever arm	$L = B - g\frac{p}{2}$	=	7.8 ft.
Weight moment	$M\,W = 15{,}750 \times 7.8$	=	122,850 ft. lbs.
Sliding safety factor	$S\,s\,f = 15{,}750 \div 7031$	=	2.24
Overturning safety factor	$O\,s\,f = 122{,}850 \div 35{,}155$	=	3.5
Pressure resultant	$R = \sqrt{3.5^2 + 7.875^2}$	=	8.6 tons.
Locus of R in base	$b = 0.5\,g\,p + O\,R - 0.5\,B$	=	0.43
Resultant falls in the middle third.			
Maximum pressure	$P\,x =$		1790 lbs.
Minimum pressure	$P\,y =$		1070 lbs.

In sec. III, Fig. 41, the water stands 3 feet above the spillway crest; characteristics are as in sec. 2 excepting H and h

$$H = 18 \text{ feet} \quad h = 3 \text{ feet}$$
$$P = 9{,}843 \text{ lbs.},\ L' = 5.713 \quad M\,P = 56{,}243 \text{ ft. lbs.}$$
$$W = 15{,}750 \text{ lbs.},\ L = 7.8 \quad M\,W = 122{,}859 \text{ ft. lbs.}$$
$$S\,s\,f = 1.6 \qquad O\,s\,f = 2.18$$
$$R = 9.27 \text{ tons} \quad b = 1.77 \quad P\,x = 2{,}905 \text{ lbs.}$$

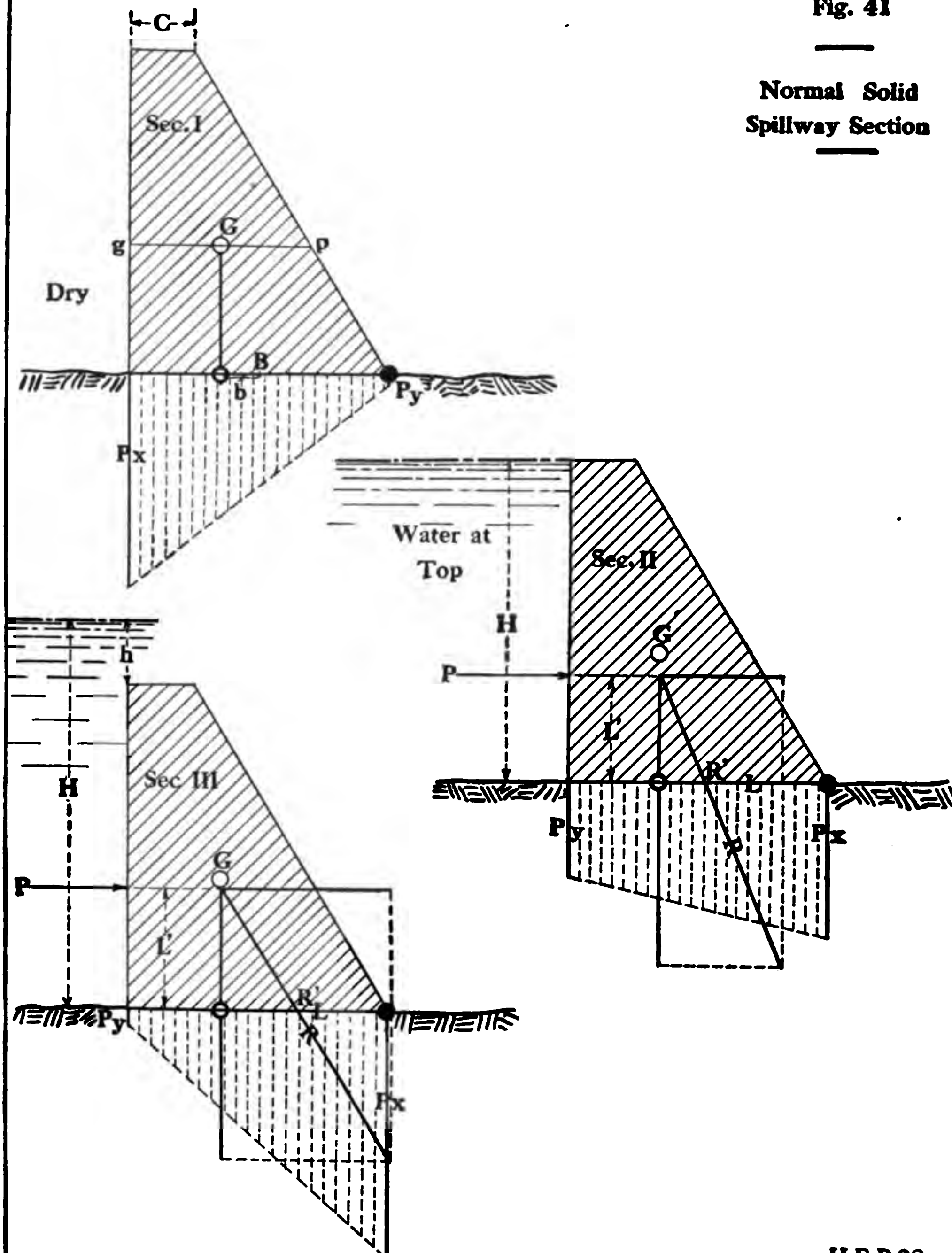
All the requirements are met by this section for a spillway of any height up to one hundred feet, provided the following conditions are complied with:

(a) the structure is properly founded and supported;
(b) protection against *underwash* is effective;
(c) the spillway consists of a homogeneous mass of *concrete;*
(d) the maximum *overflow* does not exceed two-tenths of the height of the spillway.

The *normal* section may be adapted to *abnormal overflow* by adding, for each foot of overflow in excess of the *normal,* a rectangular section *half* a foot wide to the *normal* section; and by reducing the normal section by a similar rectangle for each foot *decrease of overfall* from the normal.

Fig. 41

Normal Solid Spillway Section



H.E.P.98
H.v.S.

The *overturning* safety factor may be increased, from the normal, by the addition of a rectangle half a foot wide for each one-tenth increase of such safety factor.

Deductions and Tabulation of Dimensions, Weights, and Characteristics of the NORMAL SOLID SPILLWAY SECTION.—These expressions are to serve for the designing of CONCRETE spillways to the limit of 100 feet height; they have been equated for the two fundamental values, height of spillway and of overflow, which must be fixed before the work of designing can be approached; the expressions of weights are in *specific gravity* = 140 ÷ 62.5 = 2.24.

$$A = \frac{B + C}{2} \times S \qquad = 0.5\,S^2$$

$$*W = 2.24\,A \qquad = 1.12\,S^2$$

$$A' = \frac{H + h}{2} \times S \quad = \frac{1.2\,S + 0.2\,S}{2} \times S = 0.7\,S^2$$

$$*P = A' \qquad = 0.7\,S^2$$

$$S\,s\,f = W \div P \quad = \frac{1.12\,S^2}{0.7\,S^2} \quad = 1.6$$

$$L' = \frac{H + 2\,h}{H + h} \times \frac{S}{3}$$

$$= \frac{1.2\,S + 0.4\,S}{1.2\,S + 0.2\,S} \times \frac{S}{3} = \frac{1.6\,S^2}{4.2\,S} \quad = 0.381\,S$$

$$= \frac{6\,h + 3\,h}{6\,h + h} \times \frac{5\,h}{3} = \frac{40\,h^2}{21\,h} \quad = 1.905\,h$$

$$G\,C = \frac{B + 2\,C}{B + C} \times \frac{S}{3} \quad = \frac{0.8\,S + 0.4\,S}{S} \times \frac{S}{3} = 0.4\,S$$

$$\text{expressed in h} = 2\,h$$

$$g\,p = 0.56\,S$$

$$\text{expressed in h} = 2.8\,h$$

$$L = B - 0.5\,g\,p = 0.8\,S - 0.28\,S = 0.52\,S$$

$$\text{expressed in h} = 2.6\,h$$

$$*M\,P = L'P \quad = 1.905\,h \times 0.7\,S^2 \quad = 1.33\,h\,S^2$$

$$*M\,W = L\,W \quad = 2.6\,h \times 1.125\,S^2 \quad = 2.93\,h\,S^2$$

* Apply multiplier 62.5 to find weight in lbs.

$$O\,s\,f = L\,W + L'P \qquad = \frac{2.93\,h\,S^2}{1.33\,h\,S^2} \qquad = 2.18$$

$$*R = \sqrt{P^2 + W^2} \qquad = \sqrt{0.7\,S^4 + 1.125\,S^4} = 1.325\,S^2$$

$$b = \frac{g}{2}p + \frac{P\,L'}{W} - \frac{B}{2}$$

$$= 0.28\,S + \frac{1.33\,h\,S^2}{1.125\,S^2} - 0.4\,S \qquad = 0.118\,S$$

$$*P\,x = \frac{R}{B} + \frac{6\,R\,b}{B^2} = \frac{1.325\,S^2}{0.8\,S} + \frac{0.7\,S \times 1.325\,S^2}{0.64\,S^2} = 3.11\,S$$

TABLE 10.—CHARACTERISTICS OF THE NORMAL SOLID SPILLWAY SECTION.

$C = 0.2\,S$,	$B + C = S$, $h = C$,	$O\,s\,f = 2.2$
†$A = 0.5\,S^2$	‡ $= 0.5\,S\,(S - n)$	§ $= 0.5\,S\,(S + n)$
$A' = 0.7\,S^2$	$= 0.7\,S^2 - n\,S$	$= 0.7\,S^2 + n\,S$
$W = 1.12\,S^2$	$= 1.12\,S\,(S - n)$	$= 1.12\,S\,(S + n)$
$P = A'$		
$G\,C = 2\,h$	$= 2\,(h + 0.95\,n)$	$= 2\,(h - 0.95\,n)$
$g\,p = 0.56\,S$	$= 0.56\,(S - 0.8\,n)$	$= 0.56\,(S + 0.8\,n)$
$L = 2.6\,h$	$= 2.6\,(h + 0.9\,n)$	$= 2.6\,(h - 0.9\,n)$
$L' = 1.905\,h$	$= 1.905\,(h + 0.9\,n)$	$= 1.905\,(h - 0.9\,n)$
$S\,S\,f = 1.6$		
$S\,f = 2.18$	to increase O S f by 0.1 add 0.5 S to the section.	

ARTICLE 66.—The shaping of the crest, of the downstream face, and of the toe of the spillway are the remaining features to complete the practical design.

The crest's upstream edge should be slightly rounded, without, however, giving it an up-slope of any length, the purpose to be attained being to prevent the lodgement on or against it, during low overflow, of floatage; the square edge would be permissible were it not for the likelihood of its chipping off.

From the downstream end of this quarter-round of its upstream edge the crest should be inclined downward at about half an inch per foot.

The downstream edge of the crest should be on a curve of a radius equal to $\frac{2\,C}{3}$, the point of curve being $\frac{2\,C}{3}$ from the upstream face plane, and the point of the tangent in a horizontal plane, one foot below crest

* Apply multiplier 62.5 to find weight in lbs.

† For normal overflow; ‡ for less than normal overflow; § for more than normal overflow; n is feet of abnormal overflow.

point; this curve will be slightly fuller than the parabola of the upper film of the overfalling water, the purpose being to secure a shape of crest to which the falling water will constantly adhere in its passage over it. The same argument underlies the inclination to be given to the *downstream face*, a condition which, as will be seen, is fully met by the normal section.

Fig. 42 shows the parabola C P T in which the ordinates m a, n b, o c, etc., represent the velocity of the water $V = \sqrt{2\,g\,\frac{2}{3}\,h}$, h being the height of overflow, and the abscissæ m m, n n, o o, etc., the fall of the water, $H = 0.5\,gt^2$, H being height of fall, and t time in seconds; from this will be seen that the downstream incline of the normal section practically parallels the parabolic curve up to a point Q about $\frac{H}{4}$ above the base when the produced spillway face and the parabolic curve approach. This is the point from which the downstream face tangent should be changed into a curve of a radius equal $\frac{S}{2}$ forming the toe of the spillway, the latter having base and altitude equal to 0.1 B; the water on passing from the toe has assumed a horizontal direction and its force is spent upon its own element.

The change in area from the normal section due to the shaping of crest and of the toe is represented by the addition of 0.025 A to A.

Article 67.—*Gravity spillways* are types in which the vertical component of the water pressure is utilized as one of the resistance factors, the upstream face being inclined, resembling section VI, Fig. 38, Art. 64. The analysis of the inclined surface, triangular section, has shown that it lacks in stability when the water stands at the top of the section, because the gravity line, and consequently the resultant, falls too close to the turning point, and this can be overcome only by the addition, to the triangular section at its downstream side, of a rectangle, by which, however, the total section area becomes considerably larger than that of the normal solid spillway section. For this reason a solid section, with an upstream inclined face, is not an economical design, and in order to take advantage of the vertical pressure factor the section must be designed, in part at least, hollow.

Timber spillways have been built on this principle for many years, the upstream face being inclined 45° and flatter, but were confined to low structures only; the development of reinforced concrete structures,

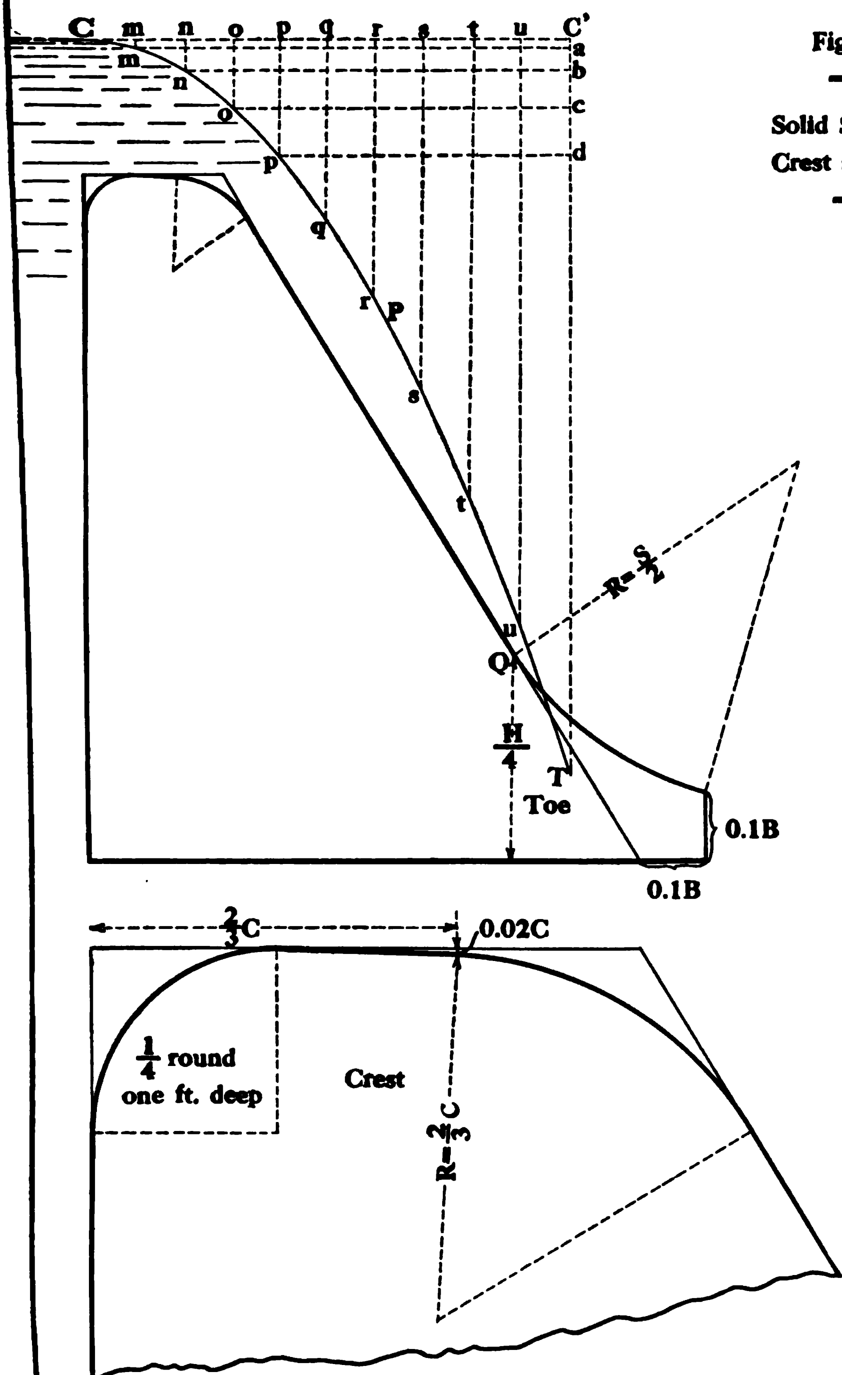


Fig. 42

Solid Spillway
Crest and Toe

H.E.P.99
H.v.S.

however, has in recent years broadened the practical application of this principle to spillway designs.

Fig. 43 shows a triangular section, I, with its upstream surface inclined 45°, to which is added a rectangle, II, of width equal to the height of the overflow, and to this a downstream triangular section, III, of such inclination that the overfalling water will adhere to it, as found in Article 66, Fig. 42.

Height of section .... S	=		15 feet
Pressed face deck .... D	= $S\sqrt{2}$		= 21.21 feet
Top or crown C	= 0.2 S		= 3 feet
Downstream side, apron ... Ap	= $\sqrt{S^2 + (0.6\,S)^2}$		= 17.5 feet
Base of sec. I .... B′	= S		= 15 feet
Base of sec. II .... B″	= C		= 3 feet
Base of sec. III .... B‴	= 0.6 S		= 9 feet
Total base .... B	=		27 feet
Overflow .... h	= 0.2 S		= 3 feet
Total height of water .... H	= 1.2 S		= 18 feet

Pressure factors:

Pressure area .... $A' = D \times \frac{H + h}{2} = S\sqrt{2} \times \frac{1.2\,S + 0.2\,S}{2}$

$= 0.7\,S^2\sqrt{2}$ = 225 sq. ft.

Pressure .... $P = A' \times 62.5$ = 14,062 lbs.

Horizontal component of P $= hP = P \div \sqrt{2}$ = 9,500 lbs.

Pressure lever arm .... $L' = \frac{H + 2h}{H + h} \times \frac{D}{3} \div \sqrt{2}$

$$= \frac{1.2\,S + 0.4\,S}{1.2\,S + 0.2\,S} \times \frac{S\sqrt{2}}{3} \div \sqrt{2}$$

$$= \frac{1.6\,S^2\sqrt{2}}{4.2\,S} \div \sqrt{2} = 5.714$$

Pressure moment .... $MP = PL'$ = 54,283 ft. pds.

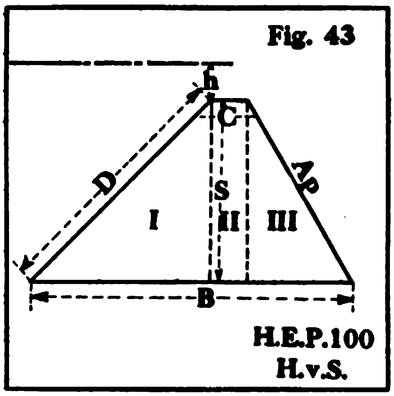
Resistance factors:

Assuming D, C, Ap to be a concrete-steel shell one foot thick, and omitting for the present the weight of the reinforcing steel:

Weight of D is	Dw = 21.21 × 140	= 2969 lbs.	
Weight of C is	Cw = 3 × 140	= 420 lbs.	
Weight of Ap is	Apw = 17.5 × 140	= 2450 lbs.	= 5,839 lbs.
Lever arm of D is	DL = B − 0.5 B′	= 19.5 feet	
Lever arm of C is	CL = 0.5 B″ + B‴	= 10.5 feet	
Lever arm of Ap is	ApL = 0.5 B‴	= 4.5 feet	

Structure's weight moment	= Dw × DL + Cw × CL + Apw × ApL	=		75,530 ft. pds.
Vertical water pressure	= hP	=	9500 lbs. =	
and its lever arm	= B − L″	=	21.37	
its moment	= vPL″	=		203,015 ft. pds.
Sliding safety factor, SSf	= W + vP ÷ hP	=	1.6	
Overturning safety factor	= WM + vPM ÷ hPM	=	5.2	
without WM, OSf	=		3.8	

and this is the distinctive principle upon which gravity spillway designs are based, namely the ratio of L″ : L′, 21.375 ÷ 5.625 = 3.8.



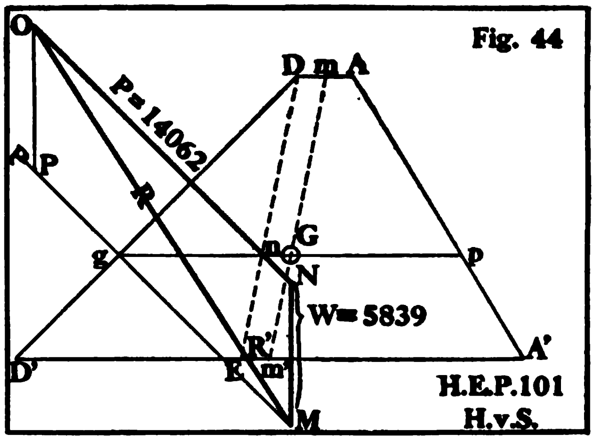


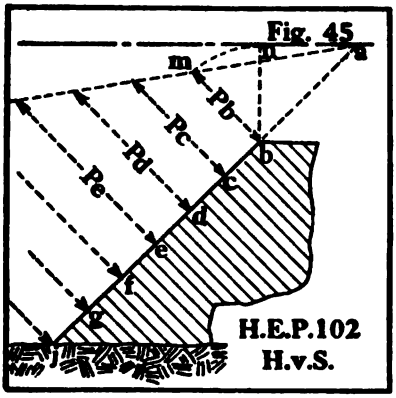
Fig. 44, *Locus of Resultant.*—The height of the statical moment GC is the same as in a solid body;

$$GC = \frac{B + 2C}{B + C} \times \frac{S}{3} = \frac{2.2S}{2S} \times \frac{S}{3} = 0.366\,S = 5.5;$$

its locus is in mm′, connecting the bisects of B and C, and is algebraically determined from

$$\begin{aligned}
DD' : Dg &= D'E : gn \\
Dg &= S - GC\sqrt{2} && = 0.634\,S\sqrt{2} = 13.44 \\
D'E &= 0.5\,B - 0.5\,C && = 12 \\
DD' &= D && = 21.21 \\
gn &= 13.44 \times 12 \div 21.21 && = \;7.6 \\
gP &= (gn + 0.5\,C)\,2 && = 18.2
\end{aligned}$$

The force diagram is projected by producing P to its intersection with the gravity line at M, laying off MN = W, and completing the parallelogram MNOP, in which the diagonal R represents the resultant of all pressure and R′ its locus in the base. Both pressure lines PM and R cut the base in its middle third.



Theoretically this gravity section is therefore safe of position. It now remains to be inquired how the parts of the structure must be designed to meet the strains to which they will be exposed. The upstream face, the deck, may fail under the superimposed weight of the water, and it must be divided into separate spans, fixed at their ends to suitable supports, when they will represent the conditions of a uniformly loaded beam secured at its ends. The bending moment on the centre of such a beam is expressed by $Mo = \frac{Q\,l}{24}$, in which Q is the product of pressure per foot into length of span in feet, l is the length of span in inches. The pressures due to the various heights of the water at different points on the deck are shown by the force lines Pb, Pc, Pd, etc. on

Fig. 45 Pb at the crown with overflow = 5 ft.

$$= b\,n \times 62.5 = 5 \times 62.5 \qquad = 312.5, \text{ also}$$

$$= a\,b \times \tan a \times 62.5$$

$$a\,b = \sqrt{b\,n^2 + a\,n^2}$$

$$b\,n = a\,n = 5 \text{ ft.}$$

$$ab = \sqrt{(b\,n^2)\,2} = b\,n\sqrt{2} = 5\sqrt{2} = 7.071$$

$$\tan a = b\,m \div ab$$

$$b\,m = b\,n = 5, \tan a = 5 \div 5\sqrt{2} = \frac{1}{\sqrt{2}}$$

$$62.5 \tan a = 62.5 \div \sqrt{2} = \qquad 44.19$$

Pb, Pc, Pd, etc., are = a b + 7 ft. sections × 62.5 tan a, or

$$Pb = (a\,b + 7) \times 44.19$$

$$Pc = (a\,b + 14) \times 44.19, \text{ etc., as per Table 11,}$$

in which P is this pressure per square foot, P_1 per square inch of one ft. of beam, and M the moment of external forces, heretofore analyzed, expressed for beams of 10, 12, and 14 feet spans.

TABLE 11.—CONCRETE-STEEL GRAVITY SPILLWAYS; OVERFLOW = 5 FEET.

Moments of Bending Forces.

a,	P	P_1	Moments for spans of 10 ft.	12 ft.	14 ft.	Spillway height.
b...........	312.5	26.0	31,200	44,928	61,152	Crown
c...........	621.8	51.8	62,160	89,510	121,834	5 feet
d...........	931.1	77.6	93,120	134,092	182,516	10 feet
e...........	1240.4	103.4	124,080	178,676	243,196	15 feet
f...........	1549.7	129.1	154,920	223,084	303,644	20 feet
g...........	1859.0	154.9	185,880	267,668	364,324	25 feet
j...........	2163.3	180.7	216,840	312,250	435,006	30 feet
k...........	2477.6	206.5	247,800	356,832	485,688	35 feet
l...........	2788.9	232.2	278,640	401,242	546,134	40 feet
r...........	3096.2	258.0	309,680	445,824	606,816	45 feet
s...........	3405.5	283.8	340,572	490,406	667,498	50 feet
u...........	3714.8	309.6	371,520	534,989	728,179	55 feet
v...........	4024.1	335.3	402,360	579,398	788,626	60 feet

The external forces must be met by the *ultimate resistance* of the material of which the beam is constructed, which, for reinforced concrete, is expressed, as per Article 53, O, by $Mo = 5505\ t^2$, in which t is the thickness of the beam in inches, provided the area of the imbedded steel, per foot of beam width, $q = 0.132\ t$.

When Mo = M, the conditions represent theoretical equilibrium, and, in order that the structure may be safe, Mo should be greater than M by a safety factor of not less than "four." To determine t, the thickness of the deck, in this case, for example at point Pb:

$$
\begin{aligned}
M &= \text{for 10 ft. span} && 62{,}160 \text{ inch pds.} \\
4\,M &= 248{,}640 && = 5{,}505\ t^2 \\
t &= \sqrt{248{,}640 \div 5505} && = 6.72 \text{ inches, and} \\
q &= 6.72 \times 0.132 && = 0.887 \text{ square inch steel}
\end{aligned}
$$

These values for t and q at points in the deck, designated as b, c, d, e, etc., applying to different heights from the crown downward, are given in Table 12 for 10, 12, and 14 feet spans.

TABLE 12.—CONCRETE-STEEL GRAVITY SPILLWAYS; VALUES OF t AND q.

Spans.	10 ft. t.	10 ft. q.	12 ft. t.	12 ft. q.	14 ft. t.	14 ft. q.	Spillway height.
b	4.76	0.628	5.71	0.754	6.67	0.880	Crown
c	6.72	0.887	8.07	1.065	9.41	1.242	5 feet
d	8.23	1.086	9.87	1.303	11.52	1.521	10 feet
e	9.49	1.253	11.39	1.503	13.29	1.754	15 feet
f	10.61	1.400	12.73	1.680	14.85	1.960	20 feet
g	11.58	1.529	13.95	1.841	16.27	2.148	25 feet
j	12.55	1.657	15.06	1.988	17.78	2.347	30 feet
k	13.42	1.711	16.12	2.128	18.79	2.480	35 feet
l	14.23	1.878	17.08	2.255	19.92	2.629	40 feet
r	15.01	1.981	18.00	2.376	21.00	2.772	45 feet
s	15.73	2.076	18.88	2.502	22.02	2.907	50 feet
u	16.43	2.168	19.72	2.603	23.00	3.036	55 feet
v	17.10	2.257	20.52	2.709	23.94	3.160	60 feet

If the supports of the deck spans take the form of partition walls, filling the entire space of the interior of the spillway shell as shown in Fig. 46, and are properly designed to take up and transmit the stresses to the foundation, as per section on Fig. 46, the quantities of the concrete and of the reinforcing steel required are those given in Table 13.

The partition walls may be constructed of x concrete, all the other parts are of xx concrete. The distribution of the reinforcing steel is also shown in Fig. 46.

TABLE 13.—CONCRETE-STEEL GRAVITY SPILLWAY; MATERIAL BILL FOR 10 FEET OF SPILLWAY ACCORDING TO THE DESIGN PER TABLE 12 AND FIG. 46.

Height, feet.	X Concrete: cub. yds.	X Concrete: 1 ft. rise.	XX Concrete: cub. yds.	XX Concrete: 1 ft. rise.	Reinf. Steel: lbs.	Reinf. Steel: 1 ft. rise.
10 feet span:						
10	7.2	1.00	8.0	1.52	1,230	146.2
20	17.2	2.09	23.2	0.99	2,690	150
30	38.1	3.25	33.1	1.06	4,190	186
40	70.6	2.90	43.7	1.16	6,050	214
50	99.5	4.25	55.3	1.22	8,190	215
60	142.0	....	67.5	....	10,340	...
12 feet span:						
10	6.9	0.97	10.3	1.43	1,600	139
20	16.6	2.11	25.0	1.12	2,990	166
30	37.7	3.24	36.2	1.07	4,650	211
40	70.1	2.89	46.9	1.10	6,760	234
50	99.0	3.58	56.9	1.13	9,100	256
60	134.8	....	69.2	....	11,660	....
14 feet span:						
10	6.5	1.12	10.8	1.70	1,760	166
20	17.7	2.14	17.8	1.30	3,430	167
30	39.1	3.21	40.8	1.31	5,100	227
40	71.2	2.59	53.9	1.42	7,370	249
50	97.1	3.31	68.1	1.54	9,860	266
60	130.2	....	83.5	....	12,520	....

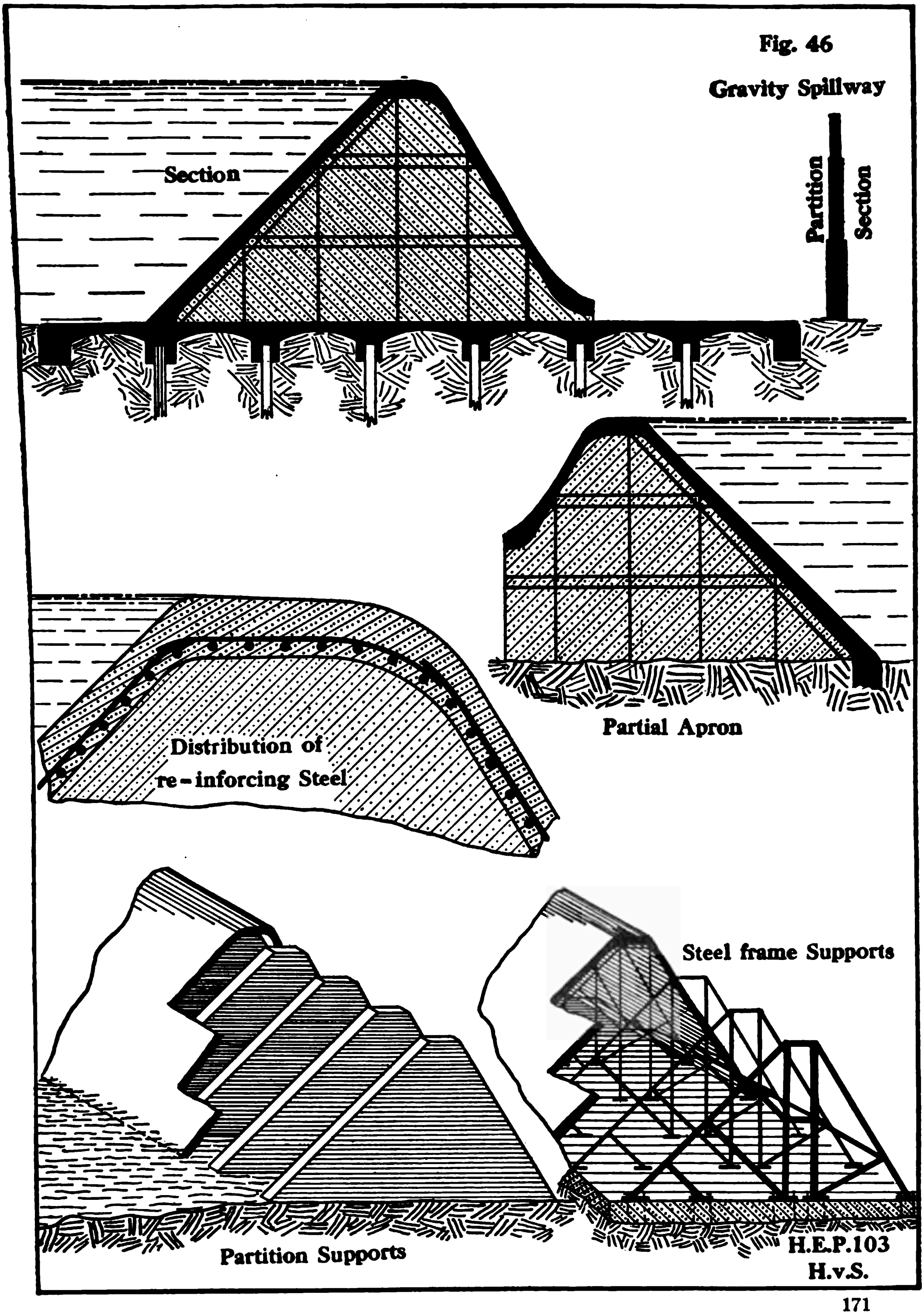


Fig. 46
Gravity Spillway
Section
Partition
Section
Partial Apron
Distribution of
re-inforcing Steel
Steel frame Supports
Partition Supports
H.E.P.103
H.v.S.

When a gravity spillway of this type is erected on a hard rock bed, its downstream face, the apron, need not be carried to the spillway toe, but may be shortened one-third or one-half, the chief consideration being whether the vertical fall of the water, passing over the spillway, from the end of the foreshortened apron face, will cause erosion of the bed rock and thereby weaken the structure's footing. A partial apron design is shown in Fig. 46; the reduction in material for this type, as compared with quantities in Table 13, consists of about 4 cubic yards of xx concrete and 600 pounds of steel for 10 feet of structure.

TABLE 14.—CHARACTERISTICS OF THE CONCRETE-STEEL GRAVITY SPILLWAY.

Deck,	$D = S\sqrt{2}$	$= 1.414\,S.$
Crown,	$C = 5$ feet	$= 5$ feet.
Apron,	$Ap = S\sqrt{1.36}$	$= 1.166\,S.$
Base,	$B = S + C + 0.6\,S$	$= 1.6\,S + 5.$

Crown is 3 feet thick.
Apron is 1 foot thick.
Partitions, from top down for each ten feet height, are respectively 12, 14, 16, 18, 21, and 24 inches thick.
Thickness of deck is as per Table 12.
Steel in deck is as per Table 12.
Steel in crown, apron, and partitions is as per Fig. 46.
The form of the crown and of the apron toe are in accordance with the theory presented for solid spillways in Article 66.

ARTICLE 68. *The Open Spillway.*—The overflow, as has been noted, is an important factor in determining the spillway section, one of its influences being to increase it in area, while the ever-present possibility of the exposure of the spillway to fluctuating pressure strains from this source is, broadly speaking, an undesirable condition. Failures of spillways, in the majority of cases, can be traced to excessive overflow; any arrangement, therefore, which allows of some control of the overflow and thereby reduces the pressure fluctuations is a very desirable one.

When the rise in the upper pool level becomes of practical utility as a power function by offsetting a corresponding rise of the lower level, thus maintaining a constant power head, this condition must be considered in arranging for the permanent lowering of the overflow.

If some of the upper portion of the spillway were removed at the approach of the high water, the excess flow could be passed inside of the spillway height, or nearly so, and the section would not have to be designed for the excessive overflow height; furthermore, the lowering of the

overflow height would materially reduce the upper pool flowage area, which will represent an appreciable economy in the cost of the development.

The effect upon the overflow by the lowering of a portion of the spillway appears from the following example.

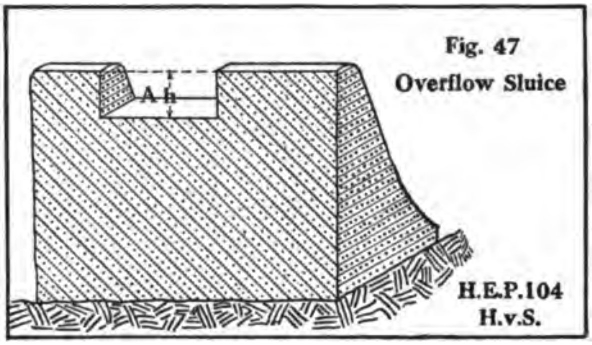


Fig. 47
Overflow Sluice

Given a spillway 200 ft. long and 30 feet high, the normal or power flow is 2000 cubic second feet and the flood discharge 10,000 sec. ft.; therefore the maximum volume which passes over the spillway is 8000 sec. ft., and the overflow is 5.2 feet.

The discharge through a rectangular opening at the spillway top, Fig. 47, is $Q = 0.62\ C\ A \sqrt{2gh}$, where C is a coefficient = 0.60, A the area of the opening in square feet, h the height of the water above the sill of the opening.

$$\text{If } h = 16 \text{ ft.,}$$
$$Q = 3\ A \sqrt{16} = 12\ A.$$
$$A = 8000 \div 12 = 667 \text{ sq. ft.}$$

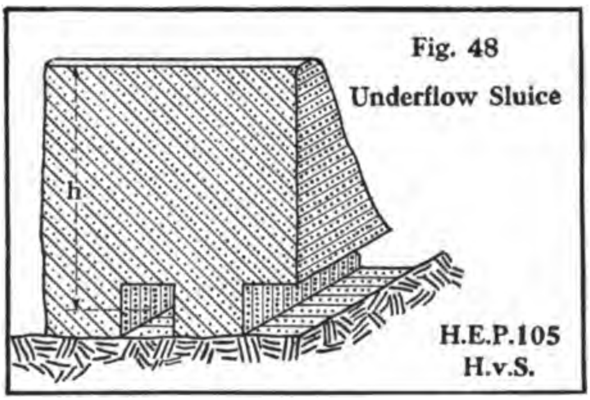


Fig. 48
Underflow Sluice

Therefore an opening of about 42 feet length and 16 feet height or two openings each 22 feet long and 16 feet high would discharge the flood flow, practically none passing over the spillway crest, and if these openings are closed during the normal flow periods the required power head may be constantly maintained.

If openings are made near the base of the same spillway the discharge through them, Fig. 48, is $Q = C\ A \sqrt{2gh}$, where C is a coefficient of discharge through a submerged orifice, A is the area of the opening in square feet, and h is the height of the water surface above the centre of gravity of the opening. For this purpose C is taken at 0.75.

If the opening is 6 feet high, the head $h = 27$ feet measured from the crest of the spillway and $Q = 6\, A \sqrt{27} = 31.2\, A$

$$A = 8000 \div 31.2 = 256.4,$$

and the flood volume of 8000 sec. ft. would be discharged through five openings each 9 feet wide and 6 feet high. The theory of efflux from orifices is developed in Article 74.

This is the theory upon which the design of the open spillway is based. The devices by which this result can be realized may be classified as

(1) *Overflow sluices*, being separate openings in the top of the spillway;

(2) *Underflow sluices*, which are similar openings at the base of the structure, and

(3) *Movable weirs*, representing continuous openings along the top of the spillway.

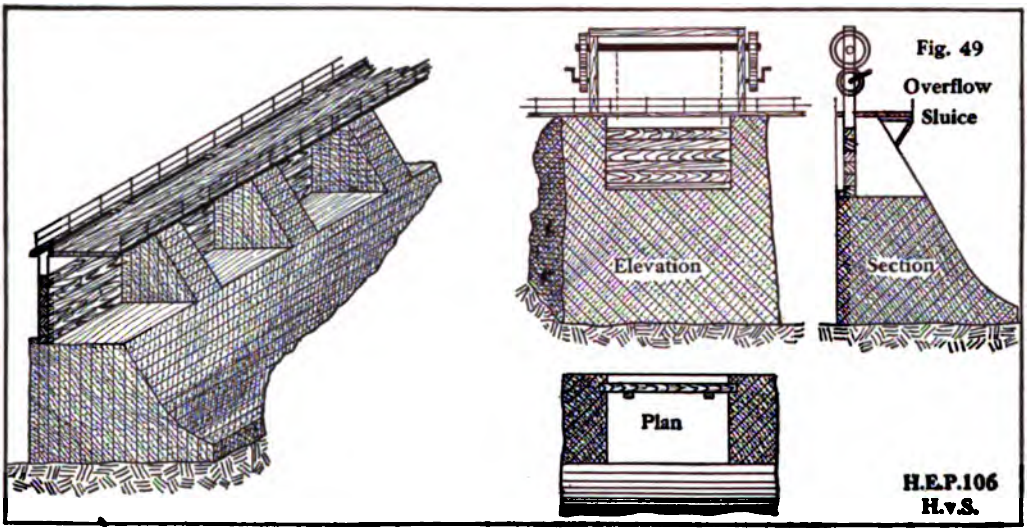
Any of these must be arranged to be readily opened and closed; they should form a water-tight wall of safe strength when closed, be simple of construction and operation and economical in cost and maintenance.

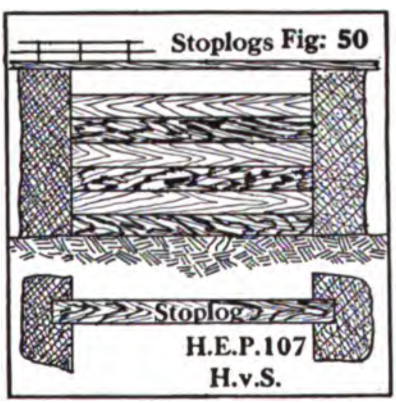
The closing devices may be classified as stop-logs, drop, lift, and revolving gates, vertical and horizontal valves, needles, wickets, shutters, and bear-traps.

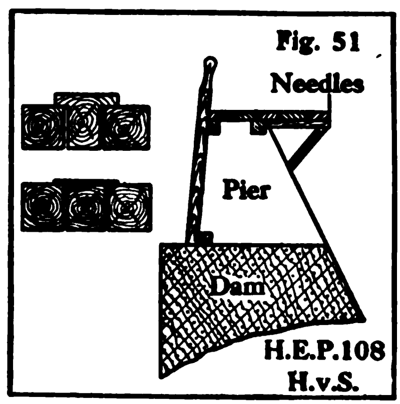
Overflow sluices, Fig. 49, are formed by masonry piers, steel or timber trestles, placed upon and secured to the body of the spillway structure; the openings, as has been shown, are determined by the volume of the water to be discharged; an operating platform is placed upon the piers or trestles. The sections of the sluice supports are so designed that the pressures and bending forces against the closed sluice spans and against their own pressure faces are safely resisted; lateral pressures need not be considered, since the accumulation of pressure heads against support sides is readily avoided.

Stop-logs, Fig. 50, may be employed to close overflow sluices; they are square timbers placed horizontally one upon another in the sluice span, their ends resting against shoulders arranged in the supports; they are operated by hand or mechanical devices from the operating platform, and can readily be formed into a water-tight curtain.

Needles, Fig. 51, are stop-logs placed vertically side by side, footing against a sill secured in the sluice bed, and supported on the top by horizontal strain members or the platform. They are not as readily manipulated as are stop-logs of the horizontal type, and it is also more difficult to make them water-tight.



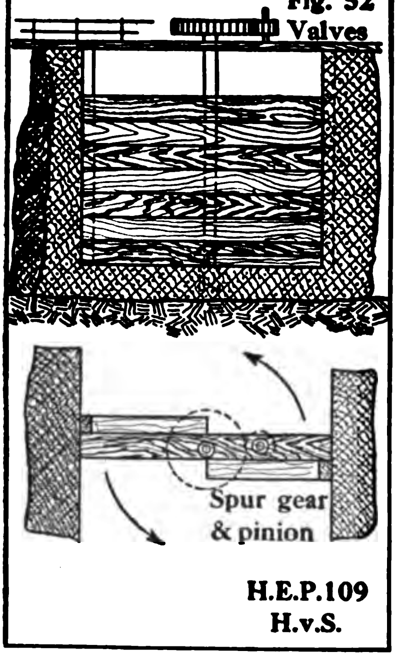




Valves, Fig. 52, are timber or steel framed shutters revolving around vertical or horizontal steel shafts, their ends set in side supports or in sluice bed and top strain member, and they close up against the sluice supports and sill; they are operated by hand or mechanically, but it is difficult to prevent leakage along their sides and bottom.

Gates, Fig. 53, may be vertical lift, drop, or revolving; they are of timber or steel frames and sheeting and are generally operated by power. Lift and revolving gates must remain suspended above the sluice span when open, requiring extra high supports, or they may be lowered into recesses arranged in the spillway masonry; in the first case they are exposed to the wind pressure and the supports must be designed in accordance; in the latter arrangement the recesses in the spillway are likely to be filled with sand. Gates form effective closing devices and can easily be made water-tight.

Fig. 52
Valves



Shutters, Fig. 54, are constructed of timber or steel and are hinged to a sill; they may also be arranged to operate automatically by folding or dividing them into two or more parts hinged on opposite sides of the sluice bed. When water from a higher level is let in underneath them, they will rise in "A" shape, and may be maintained in that position until the water is withdrawn, when they fall down on their beds.

Bear-traps are of this general design.

Underflow sluices, Fig. 55, are openings cut out of the spillway body near its base, and may be of rectangular, square, or circular form; they may be closed by lift gates or valves operated from above.

Movable weirs, Fig. 56, are devices filling the entire top of the spillway or the greater portion of it, and consist of separate steel trestles placed transversely of the spillway and hinged to the sluice base. When down they lie in a recess formed in the spillway masonry; to erect them the first trestle is raised to a vertical position by means of chains from the abutment or pier and secured by some automatic locking arrangement; after placing a foot-bridge from this first trestle to the abutment or pier, the second trestle is similarly raised, and so on. Trestles are spaced from 8 to 16 ft. centres and their upstream faces are covered with needles. They may be lowered in the same manner as described for their raising. Shutters and bear-traps may also be utilized for movable weirs.

The results aimed at with the open spillway type are to afford ready control of the overflow, to maintain the power head as constantly as practicable, and, generally speaking, to reduce the overflow height, and to accomplish all this without any waste of the water by leakage. It is evident that the ideal flow control will be secured when the overflow height is reduced by horizontal sections of the smallest areas; simplicity and economy of the device and of its operation are of like impor-

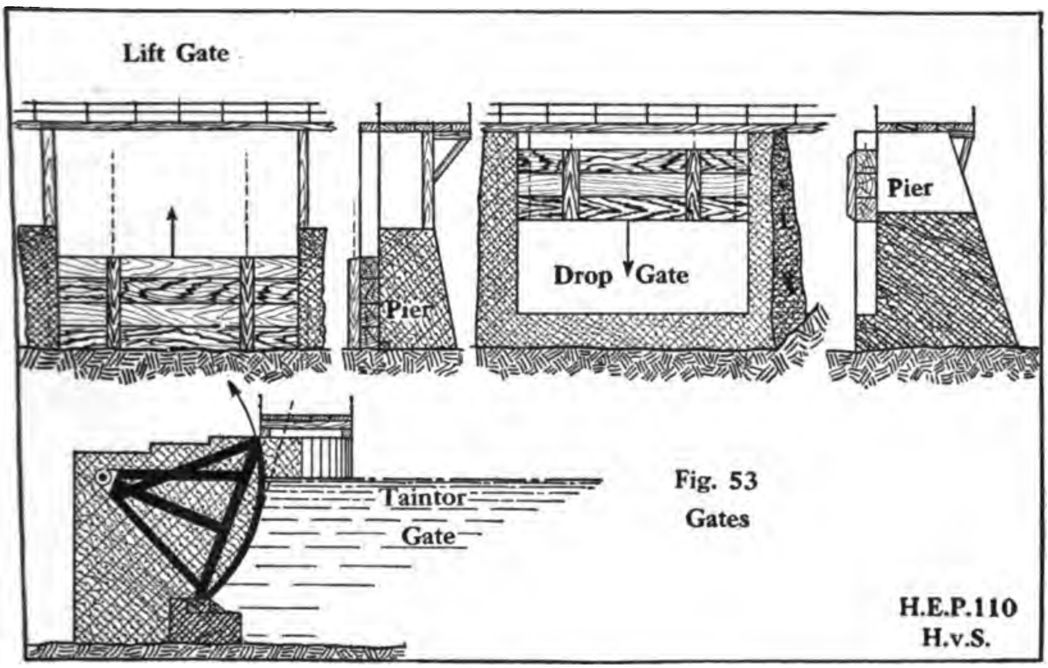


Fig. 53
Gates

tance. While the movable weir probably represents the most complete method of accomplishing this, it is also among the most costly and complicated and cannot readily be made water-tight; its field of usefulness is rather for navigation works than power plants. Both the over and underflow sluices meet the requirements, the first for the solid, the latter for the gravity spillway.

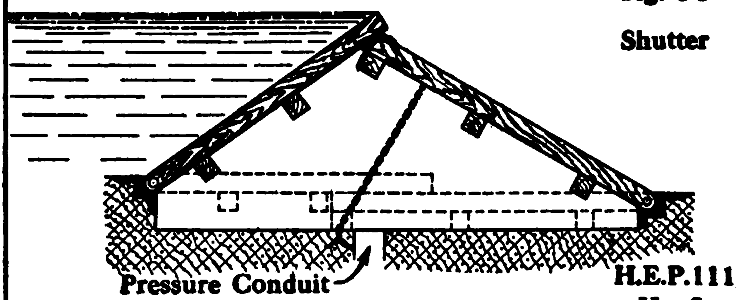
For gravity spillways the underflow sluice may be a concrete culvert or a steel plate pipe, the intake being fixed in the deck, the sluice passing through the spillway along its base and terminating at the apron toe; the flow through them can be controlled by valves operated from the interior of the spillway. In solid spillways the underflow sluice is inaccessible and weakens the structure by reducing its mass at the point where it is most required.

The overflow sluice forms the most recommendable type of the open spillway, and its closing is best arranged by stop-logs.

The sluices should be designed of rather small than large areas, as the narrower spans require shorter stop-logs which are therefore more readily handled.

Fig. 54

Shutter

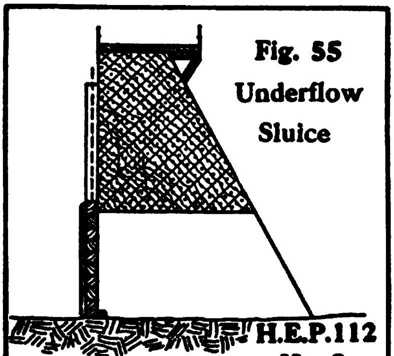


Example.—The flood flow is 3000 cub. sec. ft.; the spillway is 200 feet long and the overflow limit 2 feet. The spillway discharge aggregates 1700, and the balance of 1300 cub. sec. ft. is to be passed through overflow sluices.

The overflow sluice area required to discharge 1300 cub. sec. ft. is from $Q = 3A\sqrt{h}$ with $h = 6$, $a = 180$ (practically).

Fig. 55

Underflow Sluice



The programme is to divide the total sluice length of 30 feet into suitable units which will be taken at 10 feet; the location of these three sluices should preferably be near the end where the power station is situated, without, however, interfering with the tail-race outflow, if the station is near the spillway. The sluice supports are, most economically, concrete piers of the same section as that of the spillway, with shoulders arranged vertically in their sides to serve as stop-log supports, which may be as close to the upstream face of the spillway as practicable. The sluice pier then becomes a section of the upper six feet of the spillway section, at the base of which the length of the horizontal transverse plane

is 9.6 feet, this becomes the base of the sluice; the thickness of the pier is determined from the pressures and bending forces against the sluice span which are transmitted to the pier as the end support of a beam of the length of the span.

For a span of 10 feet length, pressure $P = 6^2 \times 31.25 = 1125$ lbs., and for 10 feet, representing one-half of the two adjacent spans, $P = 11{,}258$ lbs. For a pier three feet wide, the pressure per foot $P = 3750$, to which is added the pressure against the pier 1125; total pressure against one foot of the pier = 4875 lbs.

The depth of the sluice is taken at 6 feet and the piers should be raised about 2.5 feet to elevate the operating platform to a safe height. The pier becomes 8.5 feet high and its area .... = 59.5 sq. feet.
the weight, at 140 lbs. per cub. ft............. = 8330 lbs.
The pressure lever arm.......................... = 2 ft.,
and the weight lever arm.................... = 5.9 ft.,
Pressure moment is 9750 ft. pds.
Weight moment is 49147 ft. pds.
Sliding safety factor .... = 1.7
Overturning safety factor .... = 5
The bending moment at end supports of 10 ft. span = 112,500 inch pds.
and for the intermediate pier between two sluices .... = 225,000 inch pds.,
which is resisted by the working shearing strength of concrete taken at 75 lbs. per square inch; the pier required therefore is........... = 21 square feet;
that of the assumed section is................ = 28.8 square feet.

As a matter of fact, no economy is secured in this case, or when the required sluice length is only a small portion of the spillway length, by confining the thickness of the piers to their theoretical design, a broader pier can be constructed for less cost by permitting of the use of cyclopean concrete; but when the sluice lengths nearly take up the entire spillway, as may be the case when the spillway occupies a narrow gorge, the pier design is thus determined.

The thickness of the stop-logs is found from the bending moment at the centre of a beam supported at the ends, for this case, and the lowest beam, $P = wh = 62.5 \times 6 = 375$. The bending moment $M = Pl^2\ 12 \div 8$

$= 375 \times 10^2 \times 1.5 = 56{,}250$ inch lbs. The fibre stress of pine being taken at 2000 lbs. per square inch, $M = Fbh^2 \div 6$, b is the breadth (12″), h the thickness, therefore

$h = \sqrt{M \div 2F} = 3.8$ (appr.). In practice the log is 6″ × 12″.

The operating platform may be of 3-inch planks with guard-rails; the stop-logs can readily be raised and put in place by the aid of hand tools or by hand-power winches.

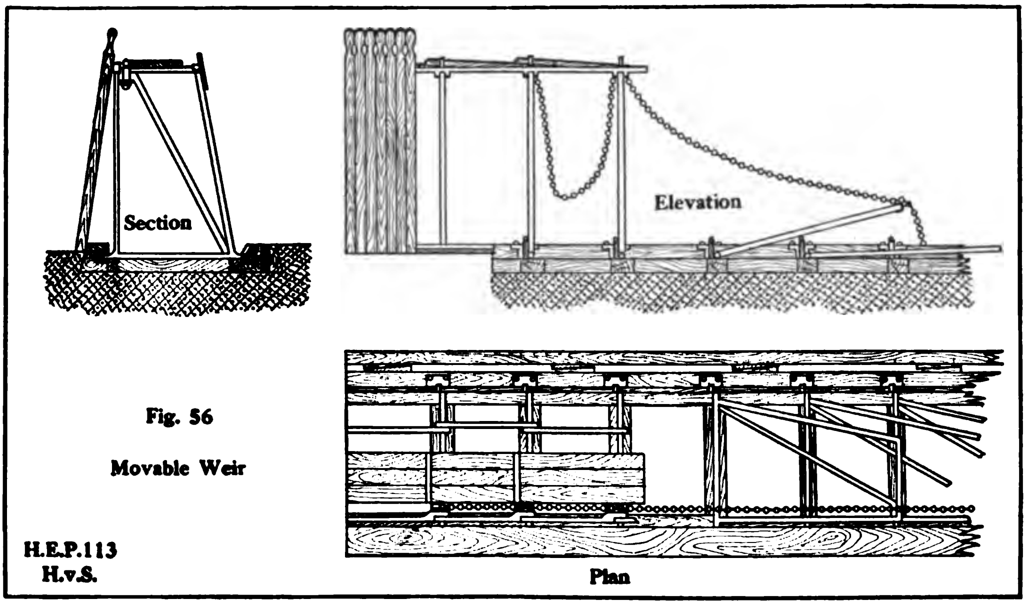


Fig. 56

Movable Weir

These represent, in the author's judgment, the three important practical spillway types, and in the light of all that has been presented regarding them the closing consideration can now be approached.

Article 69.—*In determining the spillway type* to be adopted for any specific case, adaptability, first cost, and maintenance are the weights, in their logical sequence of importance, to be applied to the inquiry; and the conditions which must guide the choice to one of these three as the best suited are the character of the river bed, the height of the spillway, and the flood volume. The river-bed formation may, for this purpose, be classified as hard and soft, rock and hard alluvial material being embraced in the first and all other alluvial composition under the second; for the former bearing foundations may generally not be required, while they are essential for any spillway in the soft locations. The spillway

height and consequential weight of the structure may have some influence, so will the volume of flood flow, the latter especially as between the open spillway and the other types.

Example 1.—A 30-feet-high spillway, 200 feet long, is to be erected in a rock river bed; the flood flow excess over the power volume is 8000 sec. ft. The solid type is adapted to these conditions, the overflow being 5.2 ft., or inside of the normal section limit, it contains 3330 cub. yds., and may be constructed of cyclopean concrete in which the ratio of manstones of 8 cub. ft. volume to concrete is about 1:3, or the material consists of about 1110 cub. yds. of manstones and 2220 cub. yds. of xx concrete. Manstones, if delivered at site for $2.50 per cub. yd., may be estimated at $3.50 per cub. yd. in place; with cement delivered at $2.50 per bbl., sand $1.00 per cub. yd., gravel or broken stone at $0.75 per cub. yd., forming timber at $25.00 per M ft. b. m., skilled labor $4.50 per day and common labor $0.20 per hour, xx concrete may be estimated at $7.00 per cub. yd. in place.

The estimated cost of the spillway superstructure then is

for 1110 cub. yds. of manstones at	$3.50	$3,885	
2220 cub. yds. xx concrete at	7.00	15,540	$19,425,

being practically $100 per lin. ft. of structure, coffering, preparing bed and foundation not being included.

The gravity spillway of the design given in Article 67 is adapted to these conditions, the overflow being only slightly above the standard therein fixed; the material contained in it for 12 ft. deck spans, according to the design of Article 67, consists of

763 cub. yds. x concrete,
724 cub. yds. xx concrete
93,000 lbs. reinf. steel.

At the same unit cost of material and labor above quoted,

x concrete may be estimated at	$10.00 per cub. yd. in place,
xx concrete may be estimated at	12.00 per cub. yd. in place, and
reinforced steel estimated at	60.00 per ton of 2000 lbs.

The estimate for the gravity spillway will then be for

762 cub. yds. x concrete at	$10.00	$7,620.00
724 cub. yds. xx concrete at	12.00	8,688.00
46.5 tons of reinf. steel at	60.00	2,790.00
		$19,098.00

or practically the same amount as the estimate for the solid type.

The open spillway for these conditions has already been detailed, and the material required in it consists of that given for the solid type less the quantities represented by the three sluice areas, or about 37 cub. yds., which, being divided between manstones and concrete at the stated ratio, makes the estimate

1098 cub. yds. manstones, $3.50	$ 3,843.00
2197 cub. yds. xx concrete 7.00	15,397.00
	$19,222.00
and for three sets of stop-logs and the operating platform	250.00
	$19,472.00

which is also practically the same estimate as the former two.

The maintenance will be very small, if any, for the solid and the gravity type, and only that for the renewals of stop-logs in the open spillway. Operating charges are alike for the first two, while the open spillway calls for constant attention, which, however, when the power station is near or at the spillway, can readily be furnished by the station personnel, but if the station is at a considerable distance from the spillway, an attendant should be located at that point.

Summing up the comparison for the rock location, it appears that each of the three types is adapted to the requirements, that the first cost of all three differs but slightly, and that neither the solid nor the gravity type involves any maintenance or operating charges, which, however, is the case with the open spillway; whether the advantage afforded by the last in controlling the flood flow, reducing the flowage areas, and removing one of the most common causes of danger to spillways is of sufficient weight to recommend it in preference to the others must be decided largely from the typical conditions of the case to be served. In this connection it is well to note that the open spillway

arrangement affords a ready opportunity to utilize the storage capacity of the upper pool in low-flow periods by accumulating water during the non-operating period of the plant.

Example 2.—The same structure is to be erected in a soft alluvial location, for which the former comparisons will prevail, with the addition of the foundation cost, which will be somewhat higher for the gravity type than for either of the other two, or practically in ratio to the width of the spillway base, which, according to the designs herein outlined, is 27 feet for the solid and open and 53 feet for the gravity spillway; the cost of coffering is also likely to be greater for the structure with wider foundation.

Example 3.—When the height of the spillway exceeds 30 feet, a new element enters the search for the most recommendable type,—that is, the location therein of the power equipment. The specific treatment of this programme will be found in the description of power stations in Article 78; suffice it to state here that for such spillway requirements, in connection with the direct development programme, the arranging of the power station in the interior of a gravity spillway is perfectly practical, and represents a considerable saving in the first cost of the plant and, in addition, secures the highest obtainable efficiency from power functions, flow and fall.

ARTICLE 70.—*Timber spillways* have played an important part in the mill-power plants of the past, and, while the advent of concrete construction and the rapid increase of the cost of timber have well-nigh discontinued their use, occasions may arise where they deserve consideration. For low-fall developments in localities where timber is plentiful while concrete material is not so, transportation facilities being limited, this type may prove recommendable because a masonry or concrete structure would be prohibitive in cost. The chief objections to timber spillways are the difficulty of constructing and maintaining them watertight and the cost of keeping them in repair. As timber is preserved best when constantly saturated, it would be preferable if water passed through and over such a spillway at all times; but when the conserving of all the flow for power purposes is a fundamental requirement of the development programme, as will often be the case, this safeguard against early decay of the timber must be forfeited and the limit of the structure's endurance becomes a factor in the determination of its availability,—that is, the maintenance cost of a timber spillway should be taken at

not less than five per cent. per annum of its first cost, whereby it could be practically renewed in twenty years.

The stability of timber spillways is determined in like manner as of the types heretofore considered,—that is, sliding and overturning must be resisted by the weight of the material with which the framed structure is filled, which may be rock, gravel, or sand, a mixture of the latter two being preferable because it represents the most compact mass. The weight of such a fill may be taken at about 94 pounds per cubic foot, or for the purpose of stability discussion at 1.5 that of water. In a crib structure of 8 feet bays the ratio of timber to filled area is about as 1 to 4.

The design of the timber spillway should be adapted to uniformity of shaping and framing the material. The upstream face may be vertical or inclined; the downstream side should conform to the inclination found to represent closely the parabolic curve of the overfalling water, but for structural reason it is best formed in steps of short threads so that floatage will not strike them; the rise of the steps should be of even number of feet to facilitate uniform framing, the threads should be of a width to make up the desired slope.

Fig. 57, section 1, suggests a timber crib spillway design in which S is vertical, B = 1.5 S, C = 0.5 B, and the stepping is arranged to form the standard overfall slope

$$S = 20 \text{ ft.}, B = 30 \text{ ft.}, C = 15 \text{ ft.},$$
$$A = 408 \text{ sq. ft., mean area of timber per lin. ft.} = 86 \text{ sq. ft.},$$
$$\text{mean area of fill per lin. ft.} = 322 \text{ sq. ft.},$$
$$P' = 0.7\, S^2 \times 62.5 = 17{,}500 \text{ lbs.},$$
$$W = 322 \times 94 = 30{,}268 \text{ lbs.},$$
$$SSF = 1.7,$$
$$L' = 24 + 8 \div 24 + 4 \times (20 \div 3) = 7.65,$$
$$MP = 17{,}500 \times 7.65 = 133{,}875 \text{ lbs.}$$

The irregularity of the timber and fill distribution renders a precise determination of L impracticable; if the body is considered as of a homogeneous mass, the form being transformed into a rectangle of similar area, or 20 feet high and 22.5 feet long, the error as relating to L will be unimportant and on the side of safety; or

$$L = 11.25, MW = 340{,}515 \text{ ft. lbs., and } OSF = 2.25.$$

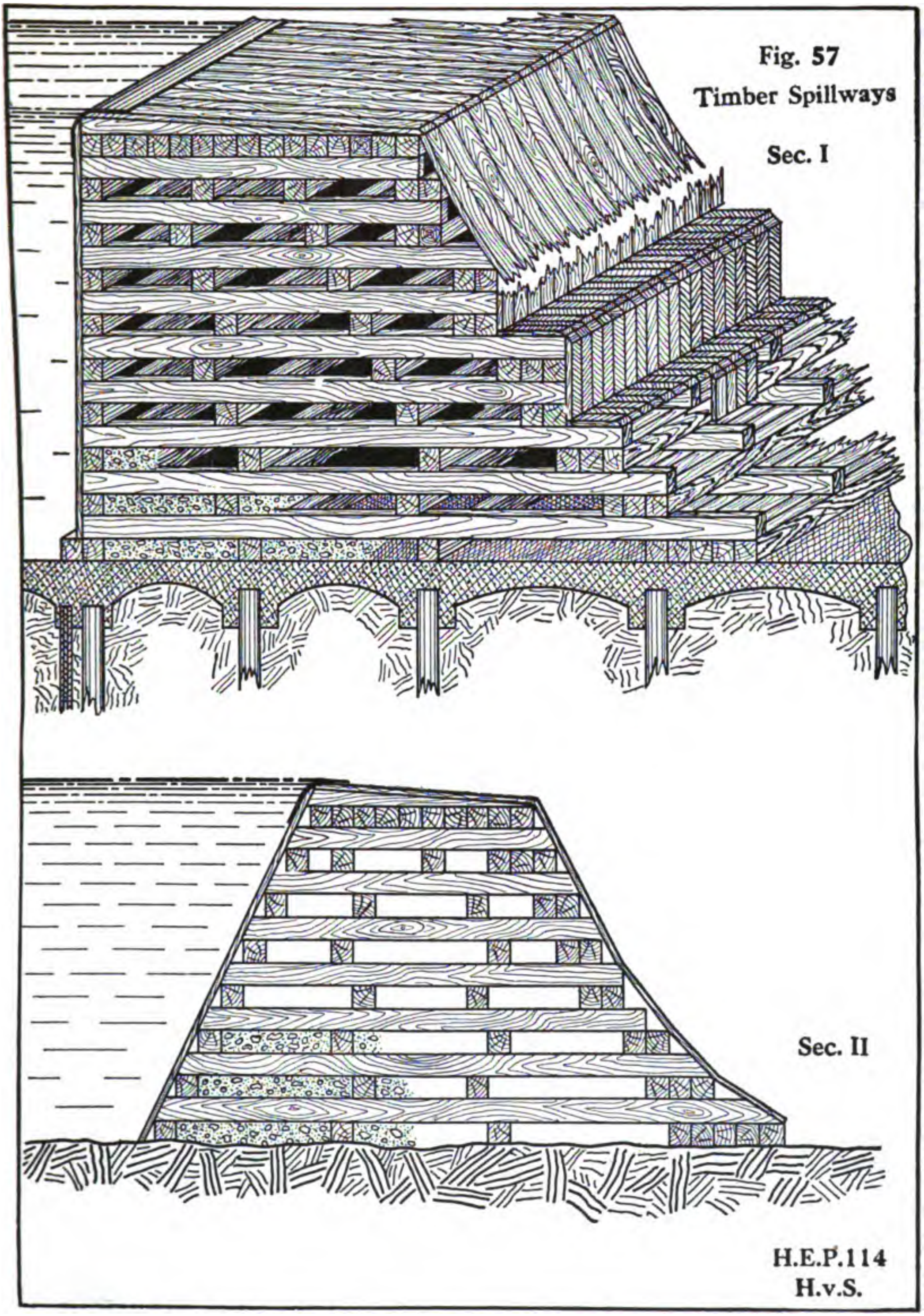


Fig. 57
Timber Spillways
Sec. I
Sec. II
H.E.P.114
H.v.S.

The framing is shown in the figure, consisting of alternate longitudinal and transverse square timbers; the former are staggered one above another excepting in the upstream face, the latter are placed 8 feet centres; the longitudinals are doubled at the apron end and are laid close in the top streak under the crown. The entire structure is covered with 3-inch planking, to make it as water-tight as possible, prevent the washing out of any of the filling material, and protect the crib timbers. The crown is given a slight slope to prevent the lodgement of floatage; its upstream edge is rounded and sheeted with iron plates.

The substructure of a timber spillway is of the same type as for concrete spillways, consisting of the cut-off, the bearing piles, if the location is in soft material, and of the apron; for the structural type of the latter heretofore described, a rock-filled trench may be substituted, that is if heavy rock is available, and the trench should be not less than three feet deep, and its width should be equal to half of the spillway height in order to insure that all of the overfall strikes this rock fill. A gravel and earth fill may be placed against the upstream side of the spillway, but it should not be expected to add to the resistance weight of the structure, as the upstream side will always be exposed to the hydrostatic head of its full height. As a matter of fact, this upstream side will sooner or later fill in from the sediment and silt carried by the stream.

TABLE 15.—APPROXIMATE QUANTITIES OF THE MATERIAL REQUIRED FOR TIMBER SPILLWAYS OF THE DESIGN HERE DESCRIBED IN LENGTHS OF EIGHT FEET AND FOR VARYING HEIGHTS.

Height.	12 × 12 timbers, ft. b.m.	Wrought iron drifts, lbs.	3 × 12 planking, ft. b.m.	Spikes, lbs.	Spillway fill, cub. yds.	Apron fill, cub. yds.
10	3,000	250	900	430	26	5
12	3,800	300	1,050	500	32	5.5
14	4,700	355	1,200	570	42	6
16	5,800	410	1,350	640	55	6.5
18	7,000	475	1,500	710	70	7
20	8,300	545	1,650	780	90	8
22	9,700	620	1,800	860	115	9
24	11,000	700	1,970	940	145	10
26	12,300	785	2,150	1,020	180	11
28	13,700	875	2,300	1,100	220	12
30	15,000	965	2,500	1,180	260	13
32	16,400	1,060	2,700	1,260	300	14
34	17,800	1,160	2,900	1,340	340	15.5
36	19,200	1,265	3,100	1,420	360	17
38	20,600	1,375	3,300	1,510	400	18.5
40	22,000	1,500	3,500	1,600	450	20

Sluices of any type may be arranged in a timber spillway. To decide between timber and concrete spillways is practically wholly a question of comparative cost and of the maintenance and repair charges.

Diagram 30 gives the approximate first cost of the timber spillway superstructure of the types here described, per linear foot of the spillway, and comparatively with the cost of concrete spillways for different heights and varying market values of timber and of concrete, both placed in the structure.

Fig. 57, section II, shows a modification of the timber spillway, the upstream side being inclined on half horizontal in one vertical and the apron fully sheeted to the shape of the overfall curve; its characteristics are: $S = 16$ ft., $B = 1.75\,S$, $C = 0.357\,B$.

ARTICLE 71.—Occasionally it may be desired to *construct the spillway at first to only a part* of the total height to be ultimately utilized; this is practicable both with the solid and the gravity types. For the first the downstream slope should then be left in steps with dowels, in order to secure the best practicable connection for the future addition to the original section; the foundation and the apron must be sufficient for the final height. For gravity spillways the partitions are likewise stepped on the apron side and are covered with a timber instead of the concrete-steel apron, the latter being constructed when the spillway is completed to its final height.

Or it may be desired to raise an existing spillway, which may also be practicable but cannot be treated in a general manner; each such case must be considered from its own conditions.

ARTICLE 72. *Spillway Abutments.*—The spillway terminates in abutments which may be of natural rock walls when the location is in a palisaded gorge; but in the majority of cases the spillway does not complete the empounding of the watercourse, and the additional structures, which are required to dam the river, are joined on the spillway, and the abutments then form the connecting links.

Abutments should be of the same height as the adjoining reservoir structures, not less than three feet above the maximum overflow level, and in outline they must be of sufficient dimensions to cover completely the ends of the banks which they are to protect against the overflowing water; they have, however, little in common with bridge abutments, partaking rather of the character of retaining walls. For solid spillways the section proper needs no abutment, which is formed around and

overlapping it on all sides as much as the section of the adjoining reservoir structures require; for gravity spillways the abutment becomes a complete wall, forming, in part, the end partition of the spillway; while for timber spillways the abutment is a separate structure throughout.

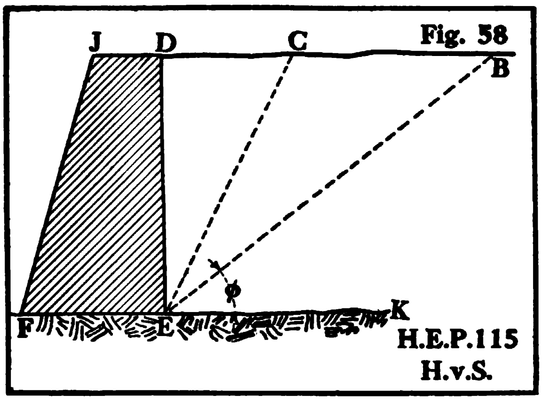


Fig. 58

The abutments may be of timber, masonry, concrete, or of concrete-steel construction, their design being based upon the theory of *earth-retaining structures*.

Fig. 58, B E K, is an earth bank, ϕ its angle of repose, B E the slope of rest; a wall, D E F J, is erected and the space between it and the bank B D F is filled with similar material as is in the bank. It is assumed that if the wall is suddenly removed a portion of the fill, represented by C D E, would follow after it, and the slope of rupture, C E, is accepted as being the bisect of an angle $= 90° - \phi$.

This theory is not well proved,—that is, as to the locus and form

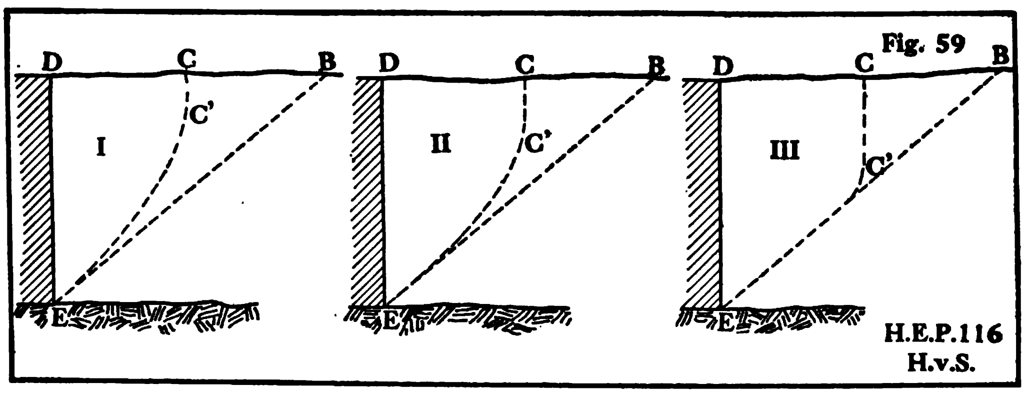


Fig. 59

of the line of rupture; it may be accepted as representing the conditions of a sand bank, provided it is dry.

The author has examined a number of slides in canal banks of from 20 to 40 feet high, chiefly of clay, gravel, and sand formation, with a view of determining the causes and effects of such subsidences, and has found the lines of rupture to be always vertical for one-third to one-half

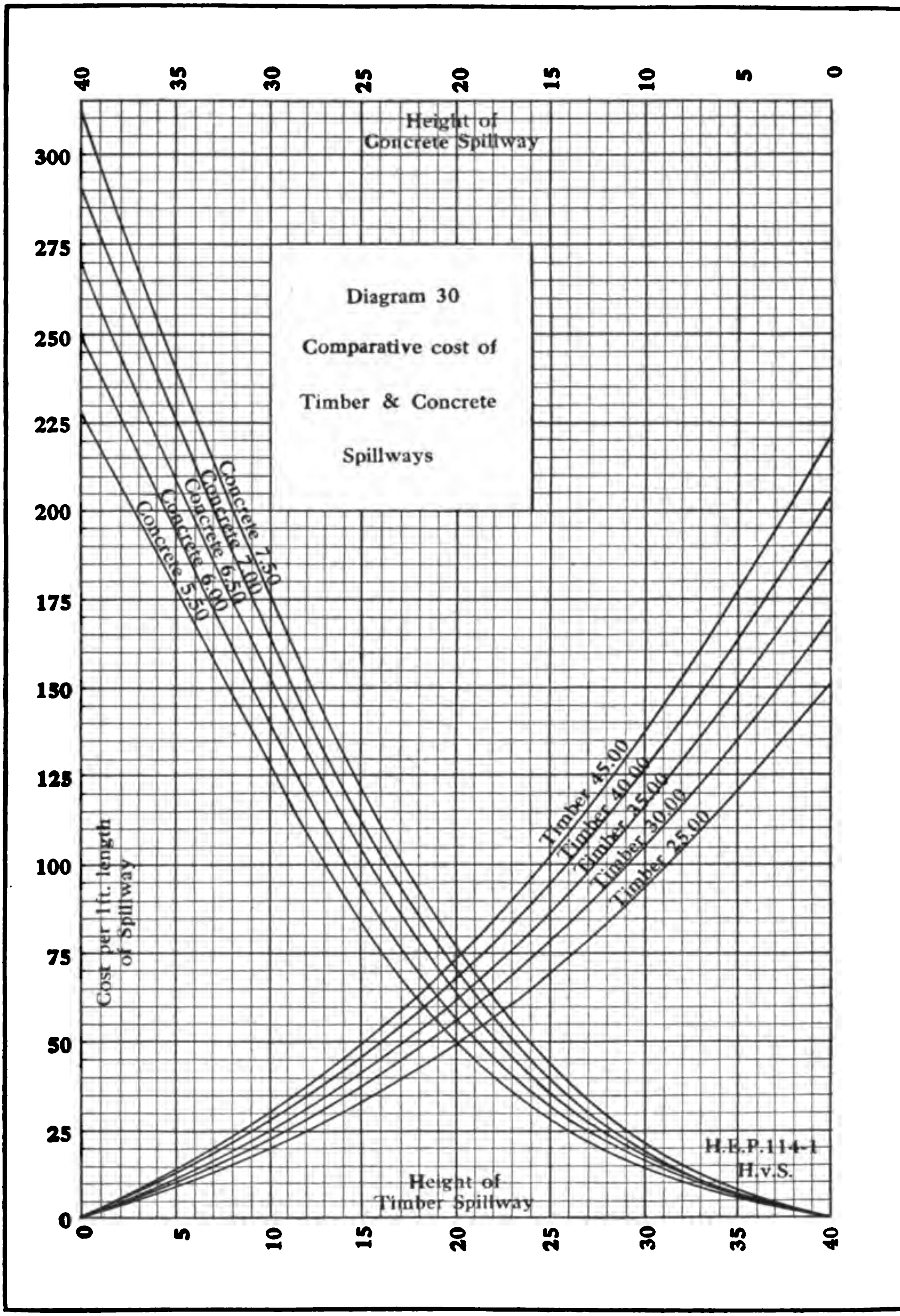
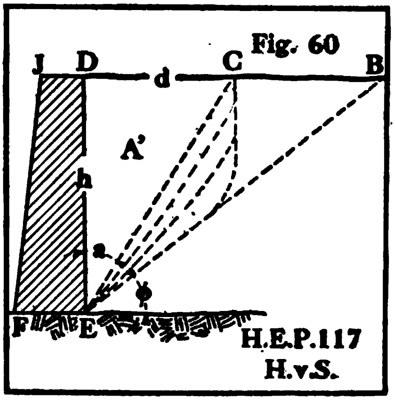
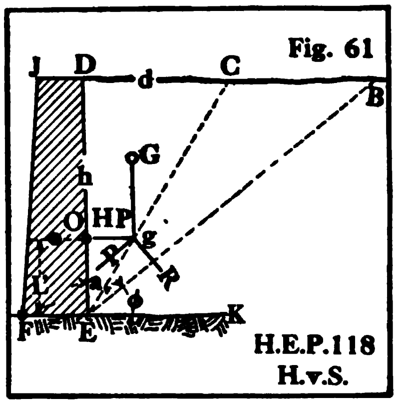


Diagram 30
Comparative cost of
Timber & Concrete
Spillways
Height of
Concrete Spillway
Height of
Timber Spillway
Cost per 1ft. length of Spillway
Concrete 7.50
Concrete 7.00
Concrete 6.50
Concrete 6.00
Concrete 5.50
Timber 45.00
Timber 40.00
Timber 35.00
Timber 30.00
Timber 25.00
H.E.P.114-1
H.v.S.
0
25
50
75
100
125
150
175
200
225
250
275
300
0
5
10
15
20
25
30
35
40

of the height, then sloping along the plane of rest, approximately as shown in Fig. 59, section I, for clay, gravel, and sand, section II, for clay and gravel, and section III, for clay. The slopes of these three, when containing only normal quantities of water, were found to be practically the same, being from 1.5 to 2.5 horizontal to one vertical; the locations and lines of rupture were likewise similar in each case, the surface break C being generally midway of DB, the top of the slide vertical and then generally curving to the toe of the slide at E, and the vertical sections increased in length as the ratio of sand and gravel decreased.





The forces to be resisted by the retaining structure originate in the wedge-shaped portion of Fig. 60, CDE, which would fall if the wall were removed; the area of this part A′ is some fraction of the rectangle hd, which will here be assumed in accordance with the observations above detailed to have the following values for different materials:

for sand or half gravel and sand or loam A′ = 0.5 hd,
for clay, gravel, and sand .... A′ = 0.6 hd,
for clay and gravel .... A′ = 0.7 hd,
for clay .... A′ = 0.8 hd,

provided that the subdrainage of the fill is sufficient to prevent any accumulation of water in the bank or against the wall.

h = height of retaining wall, and $d = h \tan \frac{90° - \phi}{2} = h \tan a$;

$A' = rh^2 \tan a$, in which r is the ratio of the area hd of the falling part as per class of material;

$W' = w'rh^2 \tan a$, in which w' is the weight in lbs. per cubic foot of the material of which the fill consists;

W' is assumed to act in the gravity line Gg of Fig. 61, on the line of rupture CE;

GR represents the reaction of the bank and

$$P = W' \times \frac{d}{h} \text{ the reaction of the wall}$$

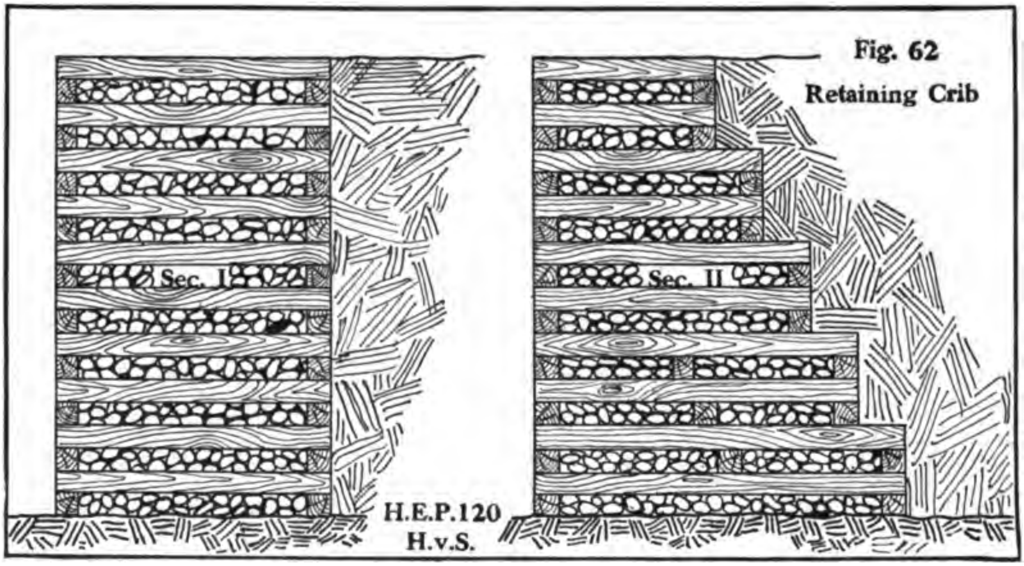


Fig. 62
Retaining Crib

$= W'rhd \times \frac{d}{h} = wrd^2 = wrh^2 \tan^2 a$ and its horizontal component $HP = P \cos \phi = w'rh^2 \tan^2 a \cos \phi$, which is the force to be resisted by the wall and is assumed to be concentrated at a point 0, being $\frac{h}{3}$ above the base EF.

The moment of pressure is the product of P into the lever arm $L' = \frac{h}{3}$

$$MP = \frac{w'rh^3}{3} \tan^2 a \cos \phi.$$

Diagrams 31, 32, and 33 give values of MP for different heights and classes of material; the weight w' and the angle of repose ϕ are those given in Article 52.

of the height, then sloping along the plane of rest, approximately as shown in Fig. 59, section I, for clay, gravel, and sand, section II, for clay and gravel, and section III, for clay. The slopes of these three, when containing only normal quantities of water, were found to be practically the same, being from 1.5 to 2.5 horizontal to one vertical; the locations and lines of rupture were likewise similar in each case, the surface break C being generally midway of DB, the top of the slide vertical and then generally curving to the toe of the slide at E, and the vertical sections increased in length as the ratio of sand and gravel decreased.

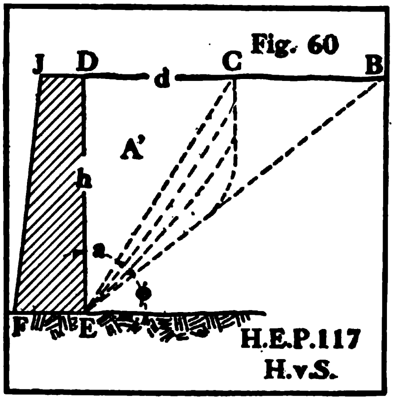


Fig. 60

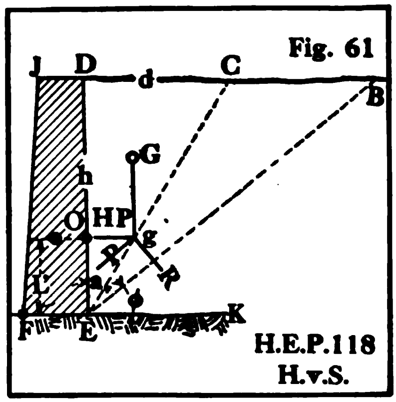


Fig. 61

The forces to be resisted by the retaining structure originate in the wedge-shaped portion of Fig. 60, CDE, which would fall if the wall were removed; the area of this part A′ is some fraction of the rectangle hd, which will here be assumed in accordance with the observations above detailed to have the following values for different materials:

for sand or half gravel and sand or loam A′ = 0.5 hd,
for clay, gravel, and sand .... A′ = 0.6 hd,
for clay and gravel .... A′ = 0.7 hd,
for clay .... A′ = 0.8 hd,

provided that the subdrainage of the fill is sufficient to prevent any accumulation of water in the bank or against the wall.

h = height of retaining wall, and $d = h \tan \frac{90° - \phi}{2} = h \tan a$;

$A' = rh^2 \tan a$, in which r is the ratio of the area hd of the falling part as per class of material;

$W' = w'rh^2 \tan a$, in which w' is the weight in lbs. per cubic foot of the material of which the fill consists;

W' is assumed to act in the gravity line Gg of Fig. 61, on the line of rupture CE;

GR represents the reaction of the bank and

$P = W' \times \frac{d}{h}$ the reaction of the wall

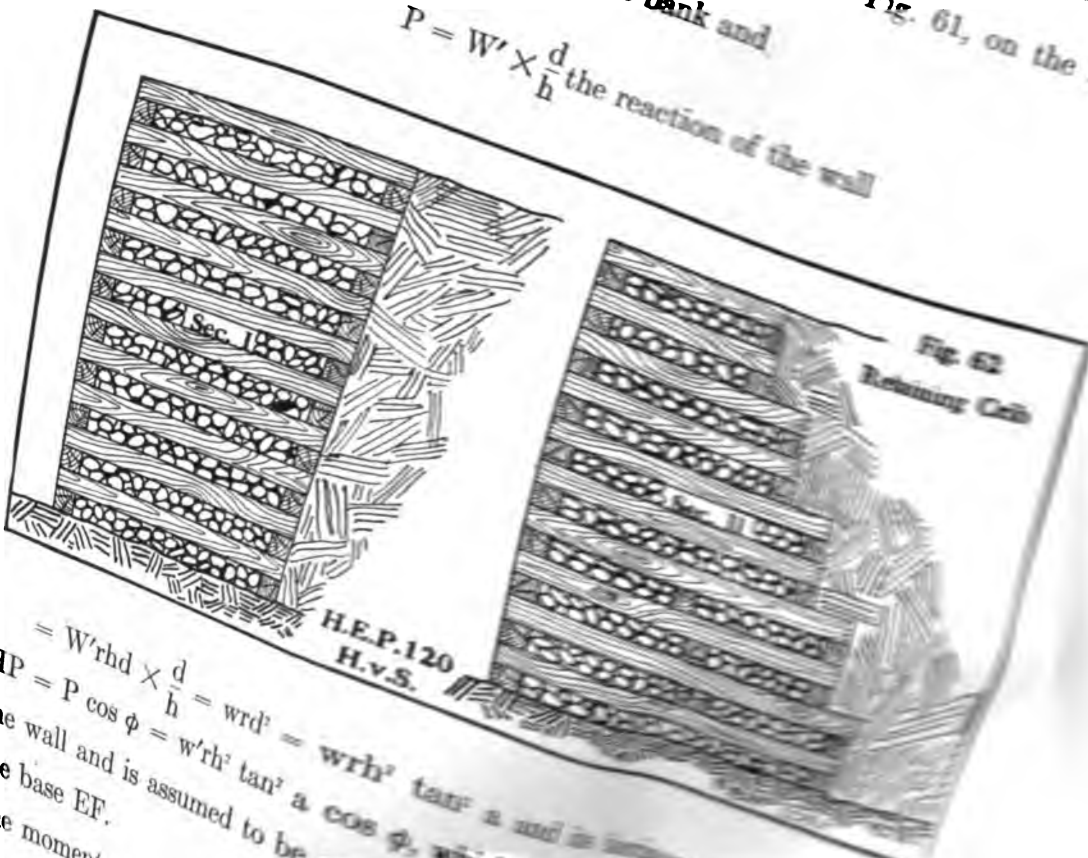


Fig. 62
Retaining Cells

$= W'rhd \times \frac{d}{h} = wrd^2 = \mathbf{wrh^2 \tan^2 a}$ and is [illegible]

$HP = P \cos \phi = w'rh^2 \tan^2 a \cos \phi$, which [illegible]

the wall and is assumed to be concentrated [illegible]

the base EF.

The moment of pressure is the product [illegible]

$MP = \frac{w'rh^3}{3} \tan^2 a \cos \phi$.

Diagrams 31, 32, and [illegible]

and classes of materia[illegible]

those given in [illegible]

The wall should be designed for a safety factor of "2" or MW = 2 MP. Ex. 1 of a timber retaining crib 20 feet high, the fill being of gravel and sand;

$$\phi = 28°,\ a = 31°,\ \tan a = 0.60,\ \cos\phi = 0.883,\ r = 0.5,\ h = 20,\ w' = 94;$$

$$A' = rh^2 \tan a = 0.5 \times 20^2 \times 0.60 = 120 \text{ sq. ft.};$$

$$W' = wrh^2 \tan a = 120 \times 94 = 11{,}280 \text{ lbs.};$$

$$P = wrh^2 \tan^2 a = 11{,}280 \times 0.6 = 6{,}768 \text{ lbs.};$$

$$HP = wrh^2 \tan^2 a \cos\phi = 6768 \times 0.883 = 5{,}976 \text{ lbs.};$$

$$MP = \frac{wrh^2}{3} \tan^2 a \cos\phi = 5976 \times 6.66 = 39{,}800 \text{ ft. lbs.}$$

Fig. 62, section I, is a crib 20 feet high and 12 feet wide filled with gravel and sand.

$$A = 240 \text{ sq. ft.},\qquad A \text{ fill} = 0.75\,A = 180 \text{ sq. ft.},$$

$$w = 94 \text{ lbs.},\qquad L = 6 \text{ ft.},$$

$$W = 180 \times 94 = 16{,}920 \text{ lbs.},$$

$$MW = WL = 16{,}920 \times 6 = 101{,}520 \text{ ft. lbs.},$$

$$SSF = 16{,}920 \div 11{,}280 = 1.5,$$

$$PSF = 101{,}520 \div 49{,}800 = 2.04.$$

Fig. 62, section II, represents the practical design of such a timber crib abutment with its bank side stepped.

Fig. 63, section I, is of a gravity retaining wall of cyclopean or monolithic concrete 20 feet high and 8 feet wide.

$$A = 160 \text{ sq. ft.},\qquad w = 140 \text{ lbs.},\qquad L = 4 \text{ ft.},$$

$$W = 22{,}400 \text{ lbs.},\qquad MW = 89{,}600 \text{ lbs.},$$

$$SSF = 2\qquad PSF = 2.25.$$

Fig. 63, section II, shows the practical design with a battered face and footing.

Diagram 32 gives quantities of material required for gravity-concrete retaining walls of different heights.

Concrete-steel retaining walls consist of a vertical concrete-steel curtain wall divided into spans of equal lengths by concrete-steel counter-

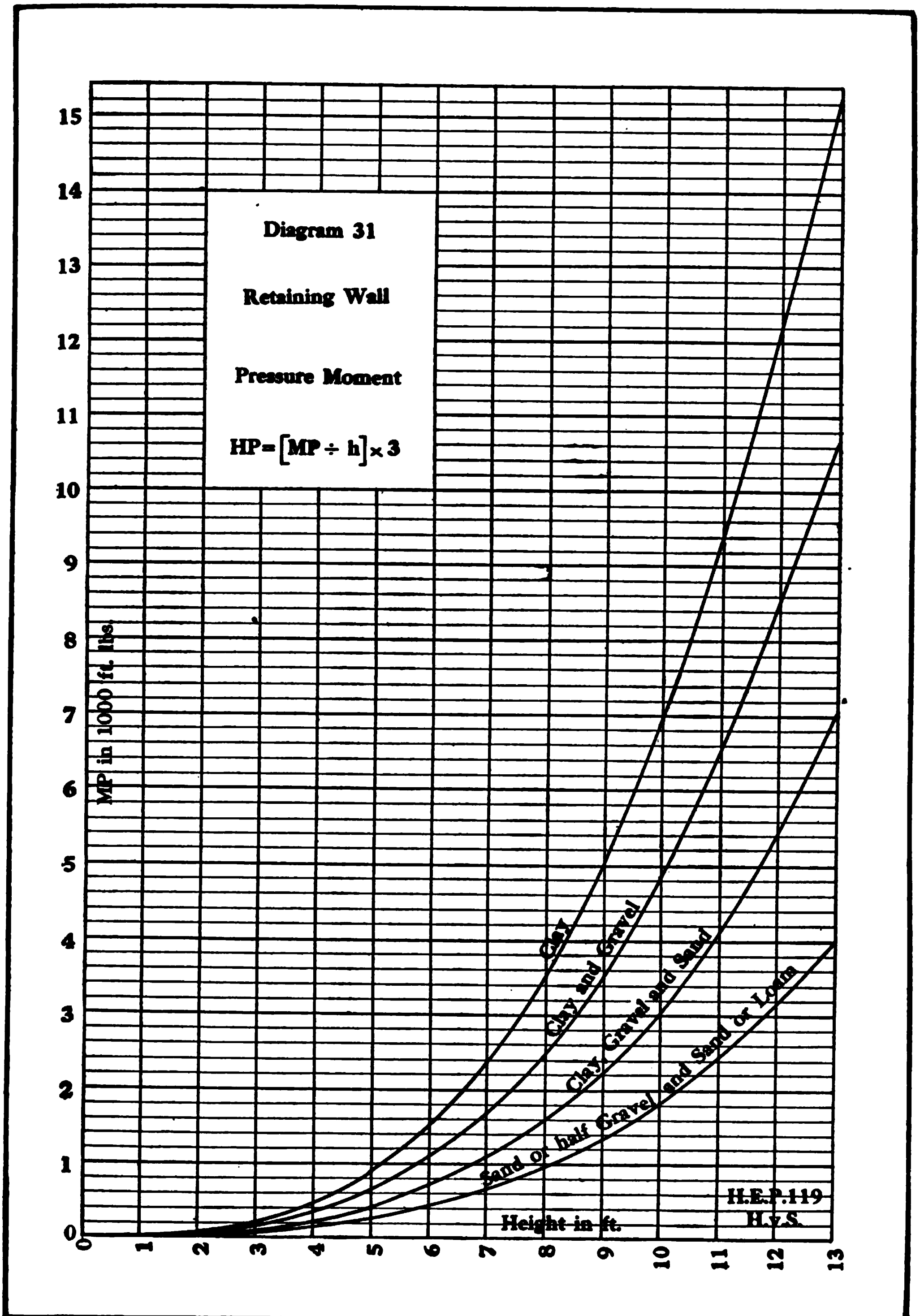


Diagram 31
Retaining Wall
Pressure Moment
HP=[MP ÷ h]×3
MP in 1000 ft. lbs.
15
14
13
12
11
10
9
8
7
6
5
4
3
2
1
0
0
1
2
3
4
5
6
7
8
9
10
11
12
13
Height in ft.
Clay
Clay and Gravel
Clay, Gravel and Sand
Sand or half Gravel and Sand or Loam
H.E.P.119
H.v.S.

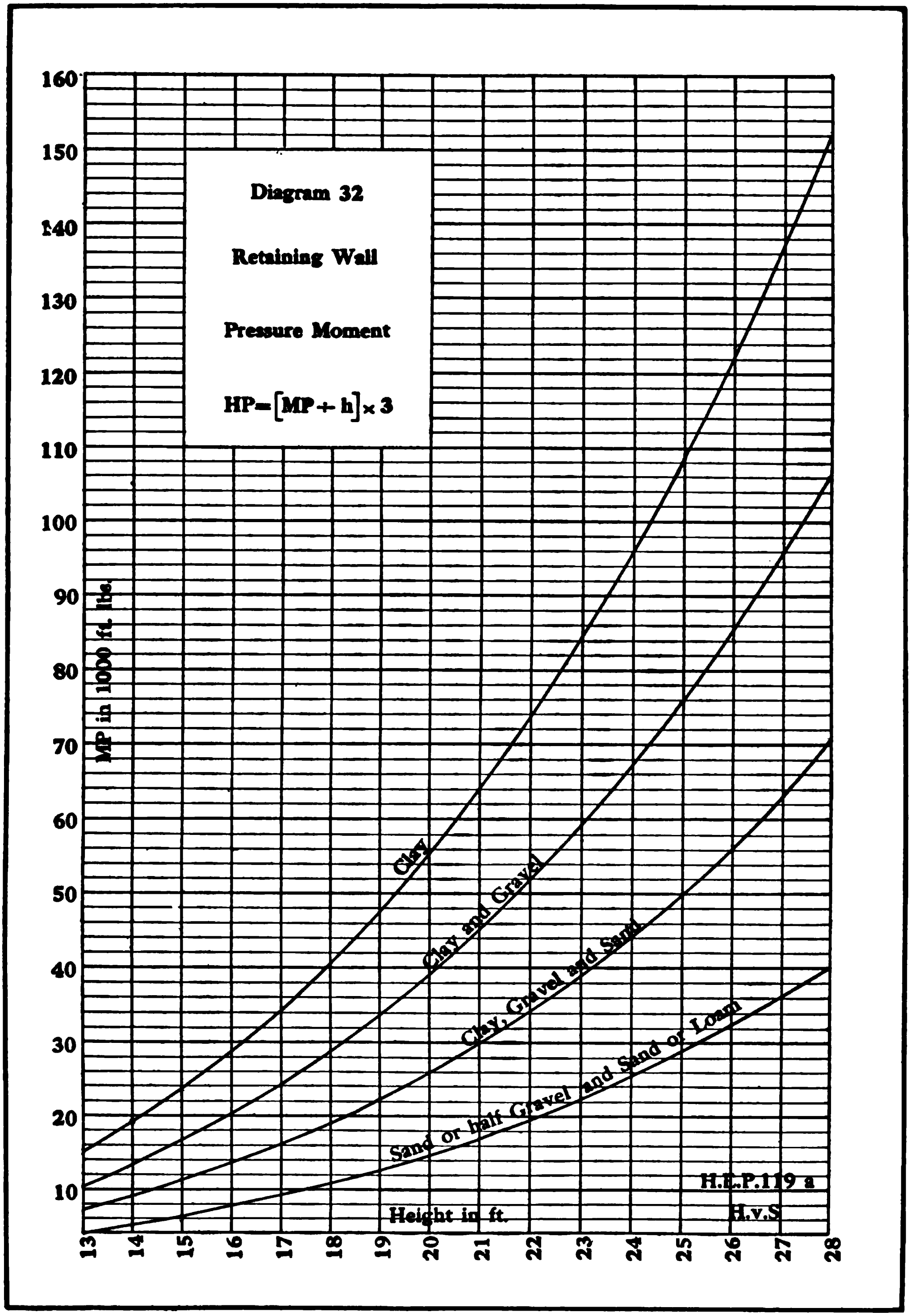


Diagram 32
Retaining Wall
Pressure Moment
HP=[MP÷h]×3
MP in 1000 ft. lbs.
160
150
140
130
120
110
100
90
80
70
60
50
40
30
20
10
Clay
Clay and Gravel
Clay, Gravel and Sand
Sand or half Gravel and Sand or Loam
H.E.P.119 a
H.v.S
Height in ft.
13
14
15
16
17
18
19
20
21
22
23
24
25
26
27
28

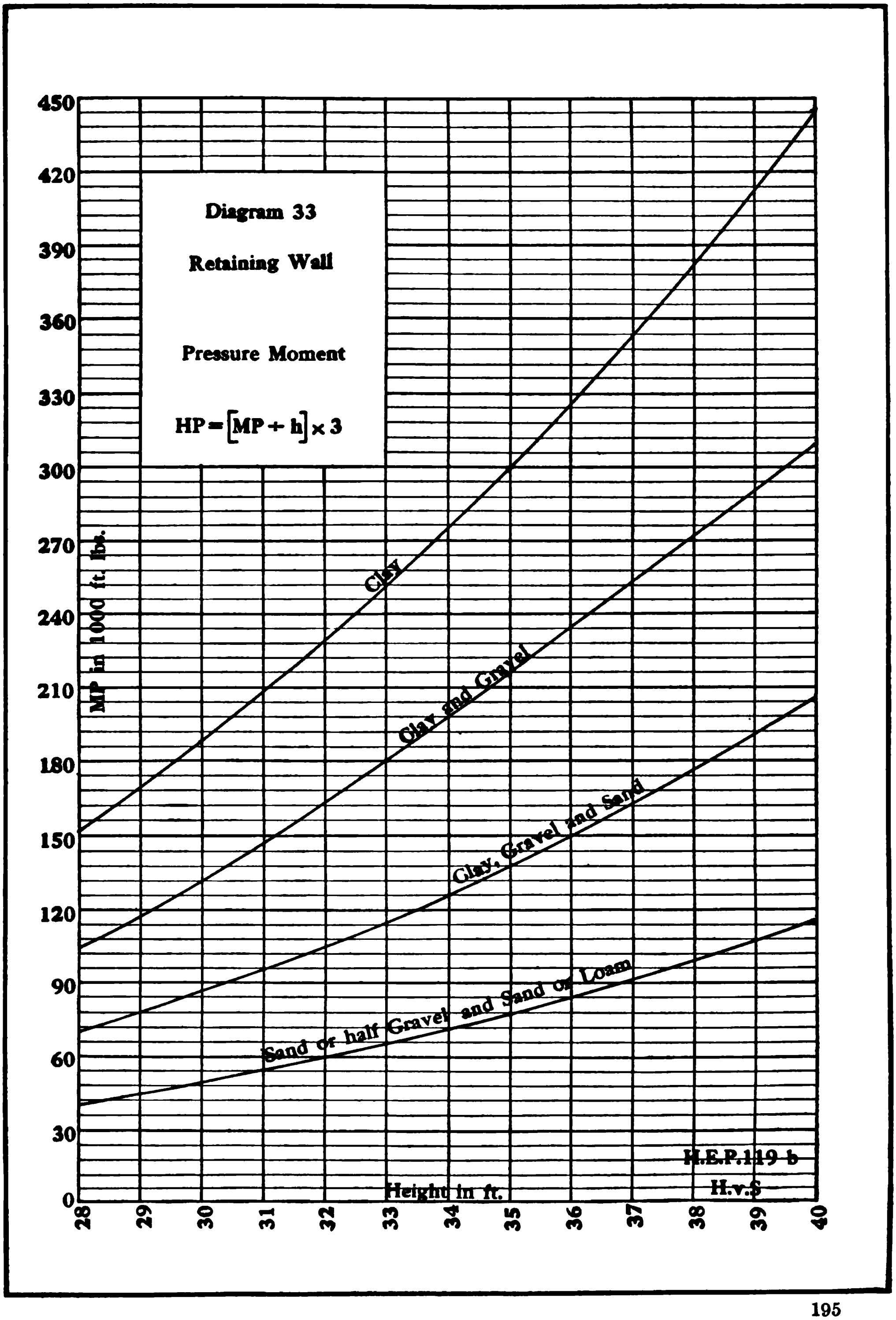


Diagram 33
Retaining Wall
Pressure Moment
HP = [MP + h] × 3
MP in 1000 ft. lbs.
450
420
390
360
330
300
270
240
210
180
150
120
90
60
30
0
Clay
Clay and Gravel
Clay, Gravel and Sand
Sand or half Gravel and Sand or Loam
Height in ft.
28
29
30
31
32
33
34
35
36
37
38
39
40
H.E.P.119 b
H.v.S

forts. The overturning forces are transmitted to and resisted by the counterforts, the curtain wall between them being designed as beams fixed at both ends and uniformly loaded.

Fig. 64, sections I and II, HP as before = $wh^2 \tan^2 a \cos \phi$; the thrust at the counterforts is

$$HPsp = wh^2 \tan^2 a \cos \phi \times \text{span, and the bending moment at FE}$$

$$Mo = \frac{w'rh^3}{3} \tan^2 a \cos \phi \times \text{span ft. lbs.}$$

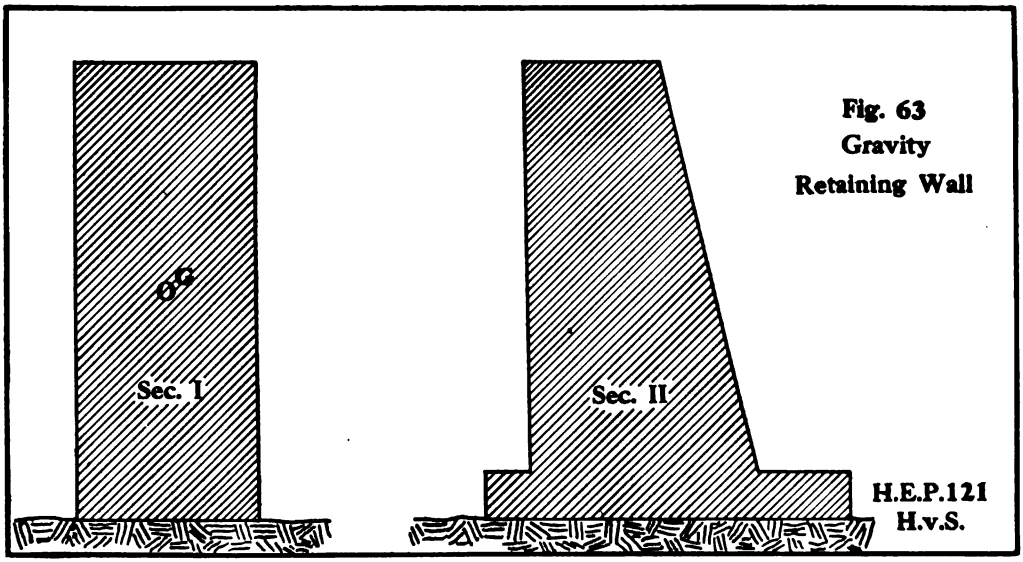


Fig. 63
Gravity
Retaining Wall

Mo4 in inch pounds, in which 4 is the assumed safety factor, is the force which must be safely resisted by the section at the base of the counterfort the top of which, at the crest of the retaining structure, may be the practicable minimum. The force against the wall between counterforts is assumed to be the uniform pressure of HP = $w'h^2 \tan^2 a \cos \phi$ acting at a point which is $\frac{h}{3}$ above the base.

Fig. 64, section III, ABC represents the pressure area acting against the wall BC.

$A' = \frac{hb}{2} = HP$; considering the wall to consist of x horizontal beams each one foot high and of the length of the span, and denoting

the vertical distance of the centre of the beam to the top of the wall by n feet, then is the load per linear foot of beam

$$W \text{ at } B = \frac{2HP}{2} \text{ and for any successive beam x from equation}$$
$$h : n = b : x, \text{ and } x = \frac{2HPn}{h^2}.$$

The bending moment against a beam *fixed* at both ends and uniformly loaded is $Mo = \frac{2HP}{12\,h} \times \frac{l^2}{12} = \frac{HPl^2}{72\,h}$; with a safety factor of 4 $Mo\,4 = \frac{HPl^2}{18\,h}$ to which the concrete-steel wall at the bottom must be adapted.

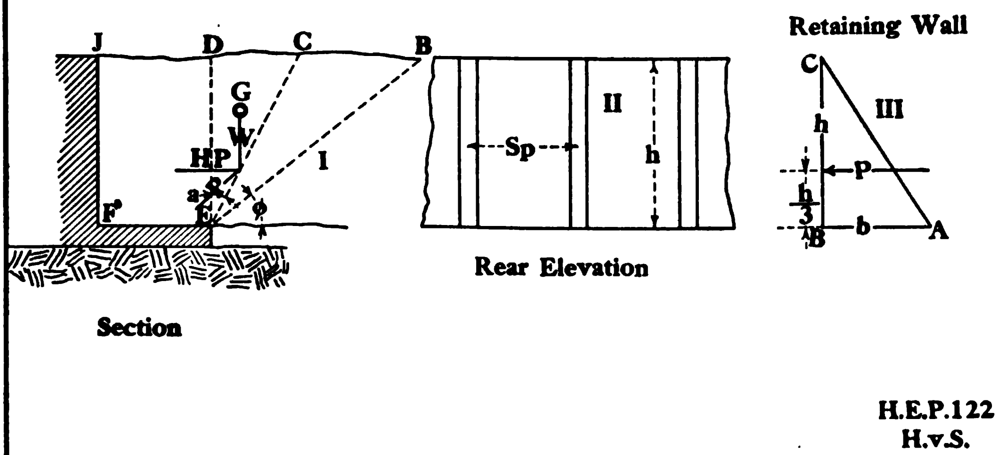


Fig. 64
Concrete Steel
Retaining Wall

Fig. 65 shows the section and elevation of a concrete-steel retaining wall. Counterforts are spaced 10 feet and are constructed of x concrete; curtain walls are of xx concrete.

$$h = 20 \text{ feet}, \qquad \phi = 28°, \qquad r = 0.5.$$

Counterforts HP = 5976 lbs.,
MP = 39,800 ft. lbs.
for a span of 10 feet MP = 398,000 ft. lbs.
Mo 4 = 19,104,000 inch lbs.,

for a section 12 inches wide the depth at the bottom of the counterfort

section "t" is, from Art. 53,

$$3620\ t^2 = 19{,}104{,}000,$$

$$t = \frac{19{,}104{,}000}{3620} = 72.6 \text{ inches},$$

and the steel area $q = 0.077\ t = 5.6$ sq. inches;

the practical section is 12 inches wide and 6 feet deep.

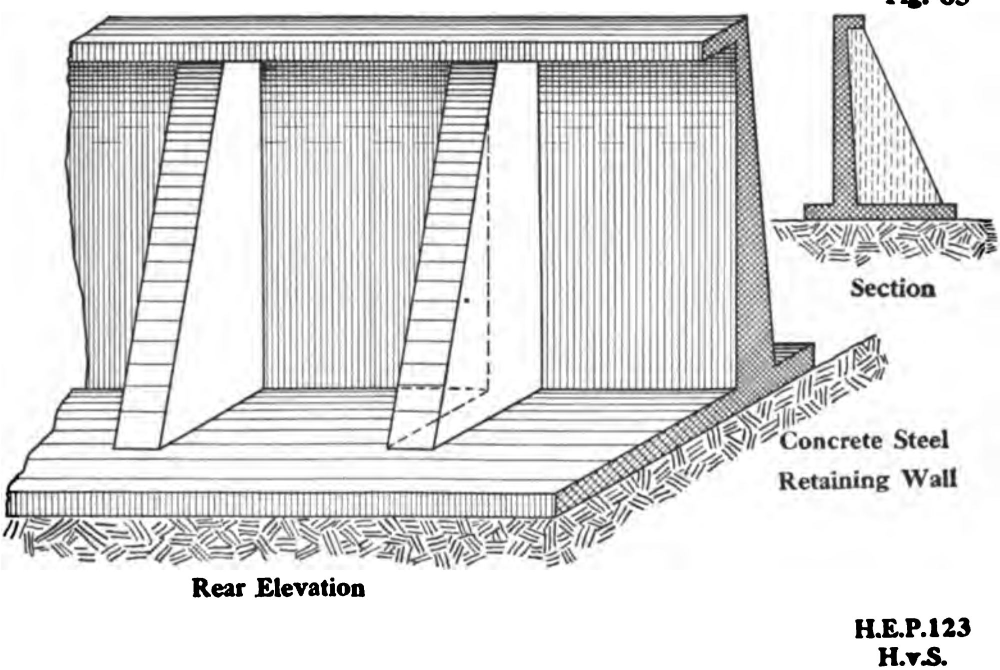


Fig. 65

Curtain wall, the bottom beam from

$$Mo\,4 = \frac{H\,P\,l^2}{18\,h} = \frac{5976 \times 120^2}{18 \times 20} \times 4 = 239{,}040 \text{ inch lbs.};$$

the actual length between counterforts is only 9 feet, but it is taken at the full span; thickness t is from

$$5505\ t^2 = 239{,}040,\quad t = \sqrt{\frac{239{,}040}{5505}} = 6.6 \text{ inches};$$

the minimum practical section of the wall will be 12 inches at the bottom and 8 inches at the top with the proper section of reinforcing steel.

Returning to the consideration of the *spillway abutment*, it has already been noted that it is a retaining wall only in part,—that is, the portion covering the spillway end is not chargeable with resistance to bank pressures which at that point are transmitted to the spillway proper; only the top and the downstream portion of the abutment, which overlap the spillway, partake of the duties of retaining walls and should be designed in accordance with the theory herein developed and as shown on Fig. 65 in plan, elevation, and section.

Approximate quantities of material required for concrete-steel abutments of various heights have been given in Part I, Diag. 12, Article 23.

The selection of the spillway abutment type is practically entirely determined by a consideration of cost; the crib abutment, like the timber spillway, will in time call for repairs, however, of no such cost and frequency as in the case of the spillway; the gravity and concrete-steel structures are both of like permanent character.

In many cases waste-flumes, as will be noted later on, are most economically arranged through the abutments, in which event the concrete-steel type has the decided advantage on account of its small transverse section as compared with the others; this is also true when the water is to be diverted from the upper pool by means of a pressure line, the intake to which is frequently most conveniently and economically arranged through the abutment.

ARTICLE 73. *Reservoir Dams.*—When the spillway completes the closing of the river valley and the empounding of the upper pool,—*i.e.*, in case the river flows between natural rock banks, which rise above the greatest flood level,—no other control works are required in connection with the dam; but where this is not the case, as in the event of the spillway taking up only a portion of the river valley, generally the natural width of the stream, and the main valley banks are at a distance from the river proper, the reach from the spillway to these banks, where they rise to the required height above the greatest overflow, must be closed by additional structures, which really form *the dam*, serving the purpose of empounding only,—that is, no flow is to pass over it at any time. These structures may be of a variety of types, generally classified as reservoir embankments and bulkheads, the first consisting of rock or earth or both, the latter of masonry, concrete, or concrete-steel.

Earth and rock-fill dams have been constructed for centuries to the greatest heights. Their design must be such that the weight represented by their section safely resists the hydrostatic pressure of the water which stands against them, that no water passes under or through them, and that their exposed slopes are safeguarded against erosion due to rainfall. The stability theories heretofore developed in connection with spillways and retaining walls also apply in this case, and, as the pressed surface of these structures is always inclined from the vertical, the pressure theories involved are specifically similar to those discussed in behalf of gravity spillways, with this exception, however, that, instead of H exceeding the height of the structure, as in the spillway, it is always less than the height of the dam, the highest water level should not rise nearer than 3 feet to the dam crest. Assuming the limits of spillway overflow as before at 0.2 of their heights, and providing this minimum clearance of 3 feet, *the height of reservoir structures* H′ will be at an elevation, referred to the footing level of the spillway, of $H' = 1.2\,S + 3$, while their actual height will depend upon the elevation of the location they occupy.

Preventing water from passing under or through a rock or earth-fill dam can be accomplished successfully only by an efficient *cut-off* below its foundation and a *core* of some impermeable material in its body. The subject of cut-off is to be treated identically as heretofore discussed in connection with the spillway foundation, and everything that has been there presented applies here; and the core is the upward continuation of the cut-off, and like it may be of various types,—namely, a timber or steel curtain, clay or concrete wall, the choice depending upon the height of the structure, availability of material and its comparative cost. The core is joined to the abutment and a drain-pipe is placed at its base on the upstream side, passing through the abutment and discharging below the spillway.

Fig. 66 shows the section of an earth and rock-fill reservoir embankment, being chiefly conditioned with a view to the preservation of its slopes, and when this is fulfilled its area and the corresponding weight are largely in excess of those required for its stability against the active pressures. The upstream slope is constantly exposed to the water, which may penetrate into the material and have a tendency to wear away the surface, which can be counteracted only by giving it an inclination of not less than two and one-half horizontal in one vertical, and provided the material of which it is formed is of the proper kind and is placed in

the manner which will be further on described. At the water surface especially will the wave action, ice, and impact of floatage make inroads into the unprotected bank, and the upper portion of this slope, for a depth covering the entire range of surface fluctuations, must be covered by a *pavement* properly laid, and in fact it is good practice to extend this pavement to one-third the water's depth. When the best material for earth dams cannot be obtained, it may be advisable and prove economical

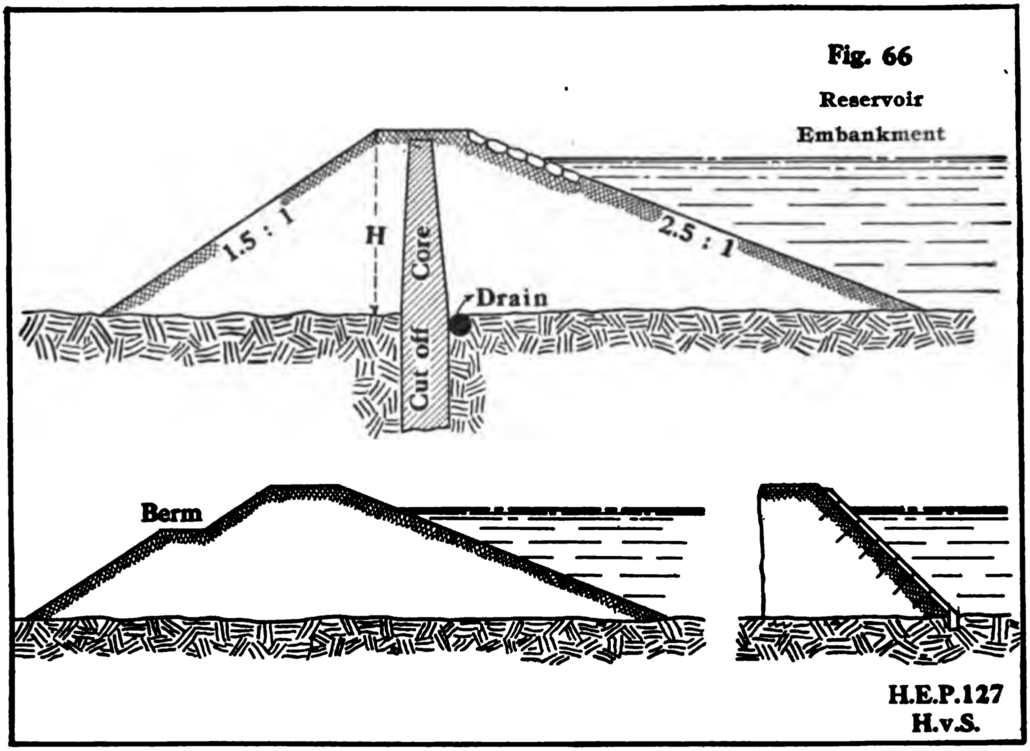


Fig. 66
Reservoir Embankment

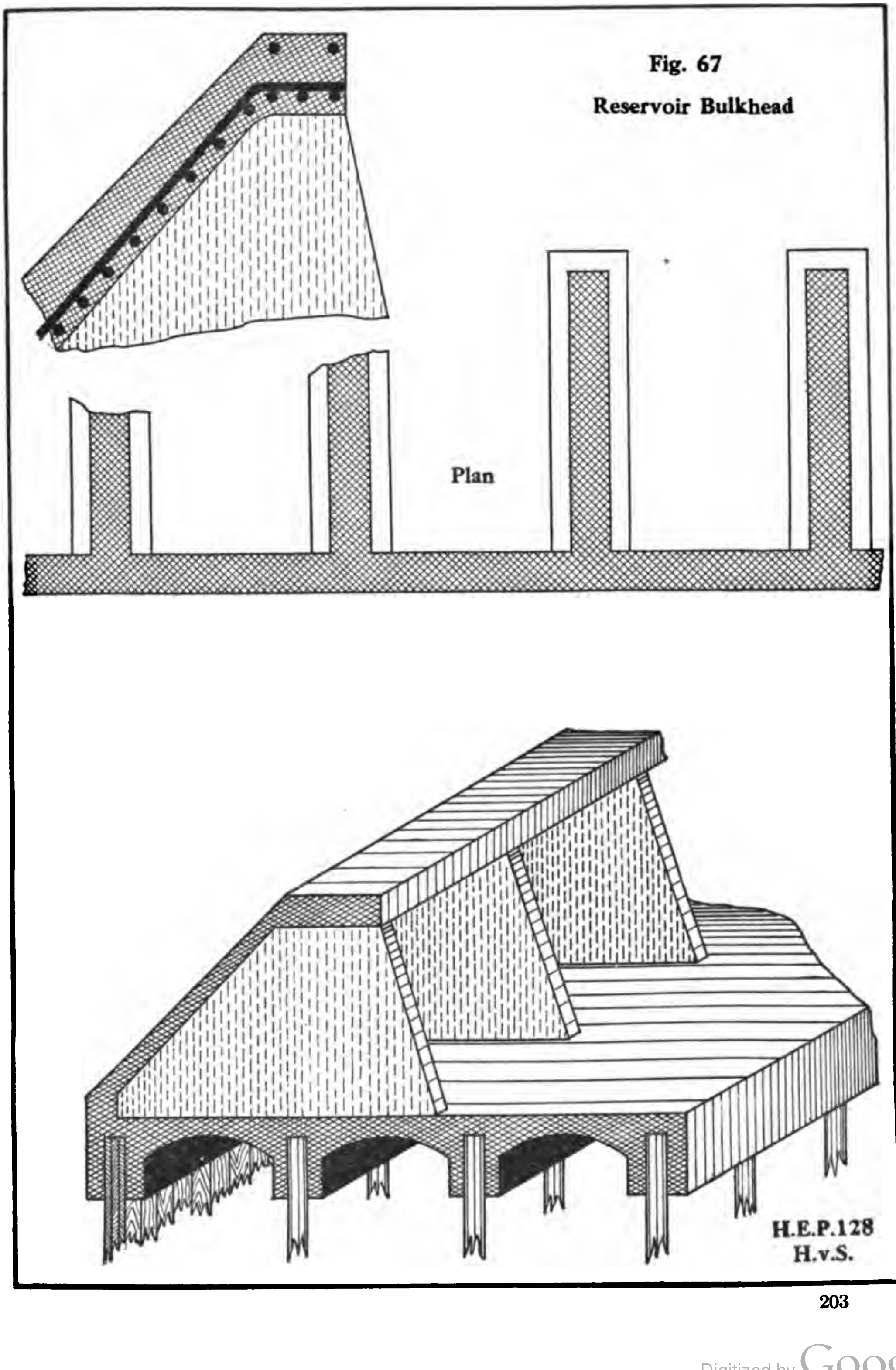
to flatten the upstream slope and cover it entirely with a blanket of concrete of a lean mix, which may be maintained in position by driving iron rods one inch in diameter and eight feet long, spaced 8 ft. c. to c., into the bank, their upset ends being imbedded in the concrete sheet; such rods are most conveniently driven by means of a pipe hammer, a five-feet long piece of inch-and-a-half iron pipe closed at one end, which is slipped over the rod and used as a driver, being exchanged for a shorter piece as the rod goes down. As the downstream slope is exposed to the rainfall, its inclination should be at least one and one-half horizontal in one vertical and it should be covered with *grass turf;* the Bermuda variety

is especially valuable on account of its long roots and thick growth. When the slope exceeds 30 feet in length, it should be broken midway by a horizontal *berme*, not less than five feet wide, to check the downflow of the run-off and thereby break its force. The *crest* should be of a least width equal to half the height and slightly rounded upward in the centre to prevent the collection of pools of water; it should preferably *not* be used as a highway, but, if serving this purpose, it should be covered with road-metal. It is a good practice to plant *trees* along the crest, as their roots will strengthen the bank; they should, however, not be set closer than five feet to the break of the crest into slope; nor should they be of rapid growth or of widely branching type which present large surfaces to the wind.

The selection of the material for reservoir embankments deserves the most careful consideration. There is a great difference between the requirements of a railroad embankment and one to maintain a reservoir; in the former proper drainage and the confining of the slope toes will generally be a sufficient guarantee for its permanency, and apparent weak places are readily accessible and can be strengthened in time; not so with the reservoir structure, one side of which is constantly submerged, where leaks are not easily traceable and are more difficult of access. The desideratum is an impervious, homogeneous mass, which may be classed as *puddle*, consisting of such proportions of gravel, sand, and clay that the voids in the mass are practically filled. Gravel, from the largest size passing a six-inch ring down to coarse sand grains, packs with 34 per cent. of its volume in voids, and if deposited in shallow layers, not exceeding six inches in depth, and covered with a two-inch layer of fine sand, the latter can be completely washed into the gravel strata, as can a succeeding layer of one inch of clay or preferably loam. This represents an expensive programme and will rarely be adopted, but is outlined here as yielding the ideal conglomerate for reservoir banks. In practice gravel, sand, and loam are spread jointly in 6 to 8 inch deep layers by means of horse-scrapers, well watered from sprinkling carts, and compacted with heavy iron rollers; all operations and handling of materials should be so arranged as to co-operate in the most thorough compacting of the mass. This treatment would be incomplete without recording a most emphatic warning against the dumping of material from cable-way or derrick buckets or from tram-cars operating on a trestle, as material so deposited *does not compact nor mix* as required for reservoir embankments.

Fig. 67

Reservoir Bulkhead



Clay should be used but sparingly; in large masses it represents the most unstable and treacherous material, and should be shunned, for the purpose of reservoir embankments, as quicksand in a railroad cut. Clay expands and contracts, in ratio of its degree of dampness, to such an extent that no bank largely formed of it, no matter of what dimensions, represents stability or permanency.

Sand, confined in place, forms an excellent bank material, and, as before stated, when combined with gravel and loam, it yields the best.

An earth embankment with a concrete core consists of two parts,—the upstream and the downstream,—the exposure and wear of which differ greatly; the former is submerged and subjected to wave action and whatever effect floatage and ice may have upon it, while the latter merely adds to the weight of the whole and is exposed to the rainfall. It appears, therefore, altogether logical that the two parts should be differently composed; for instance, the upstream section might be built of the before-described puddle, and the downstream of a loose rock-fill with sand and loam washed into the voids and a sufficient thickness of loam slope covering on the top of it to give sustenance to a grass turf. If the core-wall is of sufficient stability, such an embankment will be fully as effective as if it were formed entirely of puddle material, while, with rock available in the vicinity, its cost will be considerably less.

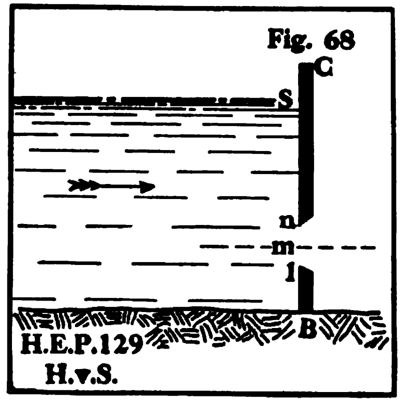
The hydraulic-fill dam is a specific type of earth dam, because of the method of its construction, which consists in washing the desired material from a higher bank, conducting it in flumes passing above the dam site, and depositing it in a semi-liquid state. By this process a very compact and thoroughly mixed mass may be secured, and this represents the best method, provided suitable material is in close proximity at such elevations that it can be sluiced by gravity for the major portion of the dam, and provided that the cost of pumping is not too great. Some high structures of this class have been erected on the Pacific coast.

Approximate quantities required for earth embankments of various heights are given in Diagram 14, Article 26.

Concrete-steel bulkheads (Fig. 67) are available for reservoir duty. They consist of an inclined concrete-steel shell supported by buttresses of like construction, their designs being based upon the same theories heretofore developed in connection with the gravity spillway, with this important exception, however, that no water is to pass over them, as they should rise to the same height as that given for reservoir embankments.

The connection of such bulkheads with the spillway is by means of concrete-steel abutments, and, in alluvial locations, they are founded upon bearing piles and concrete base with a cut-off in the manner described for the spillway. Diagram 15, Article 26, gives approximate quantities of materials required for concrete-steel bulkheads of various heights.

Fig. 68

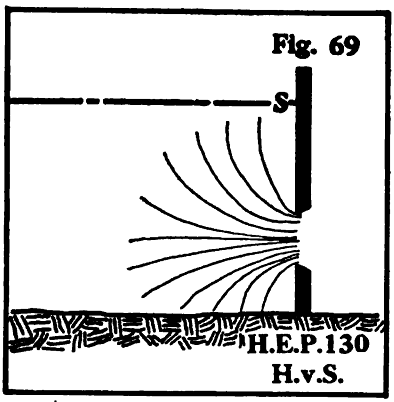


The decision as to the type of reservoir structures is, like that of abutments, chiefly a matter of cost comparison.

ARTICLE 74. *Appurtenances of Spillways and Dams.*—Provisions must be made to unwater the upper pool in order to examine and repair the dam works on that side and to remove accumulations of sediment and drift. Underflow sluices, already described in connection with the open spillway design in Article 68, are available for this purpose, and can be arranged through the spillway, the abutments, or reservoir dam, being formed of wooden stave or steel plate pipes or of concrete-steel culverts or conduits.

The theory of discharge through submerged orifices is based upon the fundamental law of flow of water, expressed by $v = \sqrt{2gh}$, v being the velocity per second, in feet, with which a body, falling freely in a vacuum, passes through the height h, g representing the acceleration of gravity in feet per second = 32.2 ft., which of course changes with the distance from the centre of gravity, the earth's centre, the above value being sufficiently correct for these requirements.

Fig. 69



In Fig. 68 CB is a partition or wall, nl is a square, rectangular, or circular opening, S is the surface level of the water; then Sl − ml = Sm is the head H under which the water passes through the opening. The theoretical discharge would therefore be expressed by Q = area of opening in square feet into v theoretical velocity, but actually both of these factors are reduced from their theoretical values by coefficients of area and of velocity.

Fig. 69 illustrates the characteristics of the actual approach of the water to such an opening, from which it is evident that a considerable part finds its way to the opening along curved paths, by which some of the energy which moves the water is lost in friction, and the theoretical velocity due to H is not fully realized when the water reaches the opening; this reduction is expressed by a coefficient of velocity which has been fixed from the results of many observations, for the practical application to this purpose, at a value of about 0.98.

Fig. 70 shows the manner in which the water enters the opening and the characteristics of its flow in its passage through the orifice, indicating the close adherence of the films of water to the full perimeter of the opening at the upstream edge, and how, by continuing for a time their curved approach directions, a distinct contraction is formed in the body of the moving water at CC′, followed directly by its gradual expansion, and by experiments carried on for long periods it has been found that

$$CC' = 0.7854 \text{ nl, and } mm' = 0.5 \text{ nl.}$$

It is apparent that CC′ represents the smallest efflux section and in it must prevail the highest velocity, which, however, cannot exceed the velocity of origin, namely that due to the head H. The greatest volume of discharge is therefore represented by that passing through the section at CC′, which is the actual volume, to wit:

$$Q = \text{area at section } CC' \times \text{theoretical velocity} \times 0.98,$$

$$CC' : ln = 0.7854 : 1 \text{ when}$$
$$ln = 1 \times 1 \text{ then } CC' = 0.7854^2 = 0.616, \text{ or when}$$
$$ln = 2 \times 5 \qquad CC' = 1.57 \times 3.927 = 6.16, \text{ or,}$$

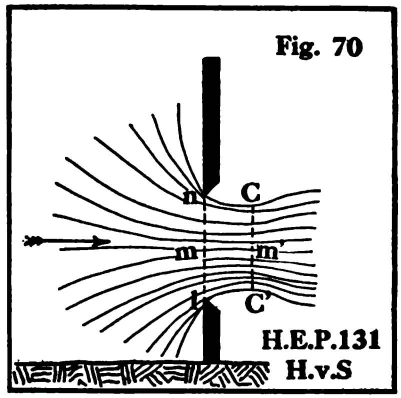
if the entrance be of circular form, the area being = $0.7854\ d^2$, and d = 1,

$$\text{then } CC' = 0.7854^2 = 0.616;$$

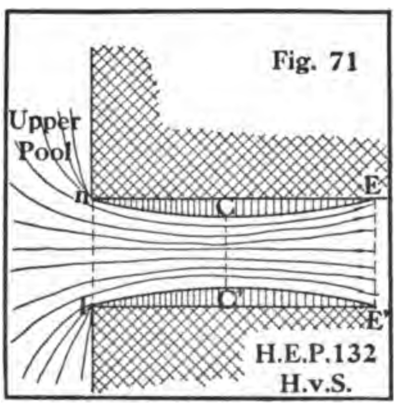
in other words, 0.61675 is the coefficient of contraction, and the ratio of the actual opening area, of whatever shape, which may be accepted as the efflux area and as the actual discharge through an opening in a thin vertical wall is

$$Q = 0.61\, a \times 0.98 \sqrt{2gh} = 0.604\, a \sqrt{2gh} = 4.85\, a \sqrt{h}.$$

In Fig. 71 the same principles are applied to an underflow sluice, nl is the entrance or intake through which the water enters from the upper pool and, as before, contracts at CC′; by this contraction, in the interior of a closed conduit, a vacuum is formed around the body of flowing water, and the latter, acting under the influence of the original energy, expands again, and, as appears from many experiments, its velocity is not reduced by reason of its flow area being increased, but the volume of efflux at EE′, the section where the entire area of the conduit is again filled with water, is greater than at CC′. This increase has been determined at about 25 per cent. over the efflux from a thin-wall opening, by which the coefficient of discharge from an underflow sluice becomes $0.61675 \times 0.98 \times 1.25 = 0.7555$, and therefore $Q = 6\,a\,\sqrt{H}$, provided the free efflux is at or near the section EE′ where the water first refills the conduit section, which point is about 2.75 nl from the intake section nl.



The same characteristics prevail if the conduit protrudes into the upper pool for a similar length = 2.75 nl, but, when it becomes longer than this, energy is expended in overcoming perimeter roughness, which in turn diminishes the velocity and therefore reduces the discharge; this enters upon the theory of "flow through pipes," which is treated in like detail in connection with the discussion of diversion works. Many refinements may be added to above outlined theory when the application is for the purpose of accurately measuring volumes by efflux from orifices, but, for the designing of works herein considered, no commensurate advantages can be secured by such reasoning, and the values are sufficiently correct for this practical use when expressed, for square and rectangular openings, by $Q = 6\,a\,\sqrt{H}$, and for circular openings, by $Q = 4.75\,d^2\,\sqrt{H}$, where H is always the head above the centre of the intake, a is area of the intake in square feet, and d is diameter of a circular intake.



The areas of the underflow sluices required to *draw down the upper pool in a given time* can be determined only if the storage volume is known; if this were A and the head maintained constant, then a quantity equal to A would pass through the sluice in $A \div 6\,a\sqrt{H}$ seconds of time, and with a constantly dropping head the time $t = 2\,A \div 6\,a\sqrt{H}$; this, however, represents only the stored volume, to which must be added the continuous flow.

Example.—Given an upper pool one mile long, 200 feet wide, and 30 feet deep at the spillway, representing a total storage volume of about 16 million cubic feet, to which is to be added the continuous low-flow volume of 1000 cubic second feet, and the pool is to be unwatered in 6 hours; then for the storage volume alone from $t = 2A \div 6\,a\sqrt{H}$, $21{,}600 = 32{,}000{,}000 \div 30a$, H being taken at 25 ft., $a = 32{,}000{,}000 \div 648{,}000 = 50$ sq. ft., to which must be added sufficient sluice area to discharge the continuous flow of 1000 sf. flow $= 1000 \times 21{,}600 = 21{,}600{,}000$ cubic feet, $21{,}600 = 43{,}200{,}000 \div 30\,a$, and $a = 43{,}200{,}000 \div 648{,}000 = 67$ sq. ft., or the total sluice area required being $50 + 67 = 117$ sq. ft., which, if arranged in two sluices, would call for an area in each of 60 square feet, and the practical dimensions would be 8×8, or if circular, each sluice would be represented by an 88-inch pipe.

In open spillways the area of sluices will always exceed that required for unwatering the upper pool, as they are proportioned to control the flood discharge of the stream.

The construction designs of sluices have been treated generally in connection with open spillways in Article 68, and so have the different gate devices.

When *sluices* are arranged *through reservoir dams* or embankments, the considerations involved do not differ from those herein discussed; the structural designs must provide sufficient safeguards against erosion of any portion of the embankment at the sluice entrance and along its location through the body of the fill.

The operation of the gates is most conveniently arranged by a well rising upward from the sluice in the downstream portion of the embankment and terminating at some convenient elevation in a gate-house.

When *log chutes* are required, they may be of the general design of overflow sluices, as described in connection with the open spillway in Article 68, with the addition of *floating booms* to intercept and guide the logs into the sluice, a sufficient *operating platform* above the log chute

from which the logs can be steered through the sluice, and, if the sluice floor is more than 5 feet above the lower pool level, of a *log apron* below the sluice, which consists of an *incline* formed of timber trestles or cribs along which the logs will pass into the lower pool without being damaged; the incline should not be steeper than 3 horizontal in one vertical and the depth of the water at the incline toe should equal half the vertical height of the apron. The log chute is best placed at that end of the spillway where the abutment can be utilized to give support to the log apron, and, if its operation does not interfere with the diversion or power station programme, the power end of the spillway is the preferable one.

In many States the law requires the placing of *fish-ladders* in any structure which obstructs the normal flow of streams, in order to enable the migrating species to reach the upper parts of the watercourse during the spawning periods. The design for fish-ladders may be selected from a variety of types, generally consisting of a trough leading from the spillway crest, at a shallow overflow sluice, to the lower pool. The inclination of the fish-ladder should be from 8 to 10 horizontal in one vertical, its width not less than 6 feet, and the depth such that water will stand not less than 18 inches deep in the steps of the ladder as shown in Fig. 72, where the arrows indicate the leaping or swimming passage of the fish. The structure is best placed at a spillway end where it may find support at the abutment, which latter is to be extended sufficiently downstreamward; the ladder proper may be supported on timber or steel trestles or on masonry piers, the choice depending chiefly upon the exposure of the ladder to the overflowing water. In open spillways, where the overflow is under control, fish-ladders may be of timber construction, with proper consideration for winter conditions; in connection with solid spillways of considerable overflow, fish-ladders should be of masonry construction. For high spillways the ladders are best arranged in several flights.

Ice-Fenders.—In northern latitudes where ice forms 6 inches and thicker, the resultant thrust against the spillway can be greatly diminished by securing ice-fenders to the upstream face of the solid spillway; these consist of two or more 12-inch square timbers placed as stringers one upon another with staggered joints and secured to the spillway by means of screwbolts set in the masonry; the top stringer is flush with the spillway crest, with an up-slant so that ice will rise up under pressure.

Flashboards are devices by which the upper pool level is temporarily raised for the purpose of accumulating an increased volume of water

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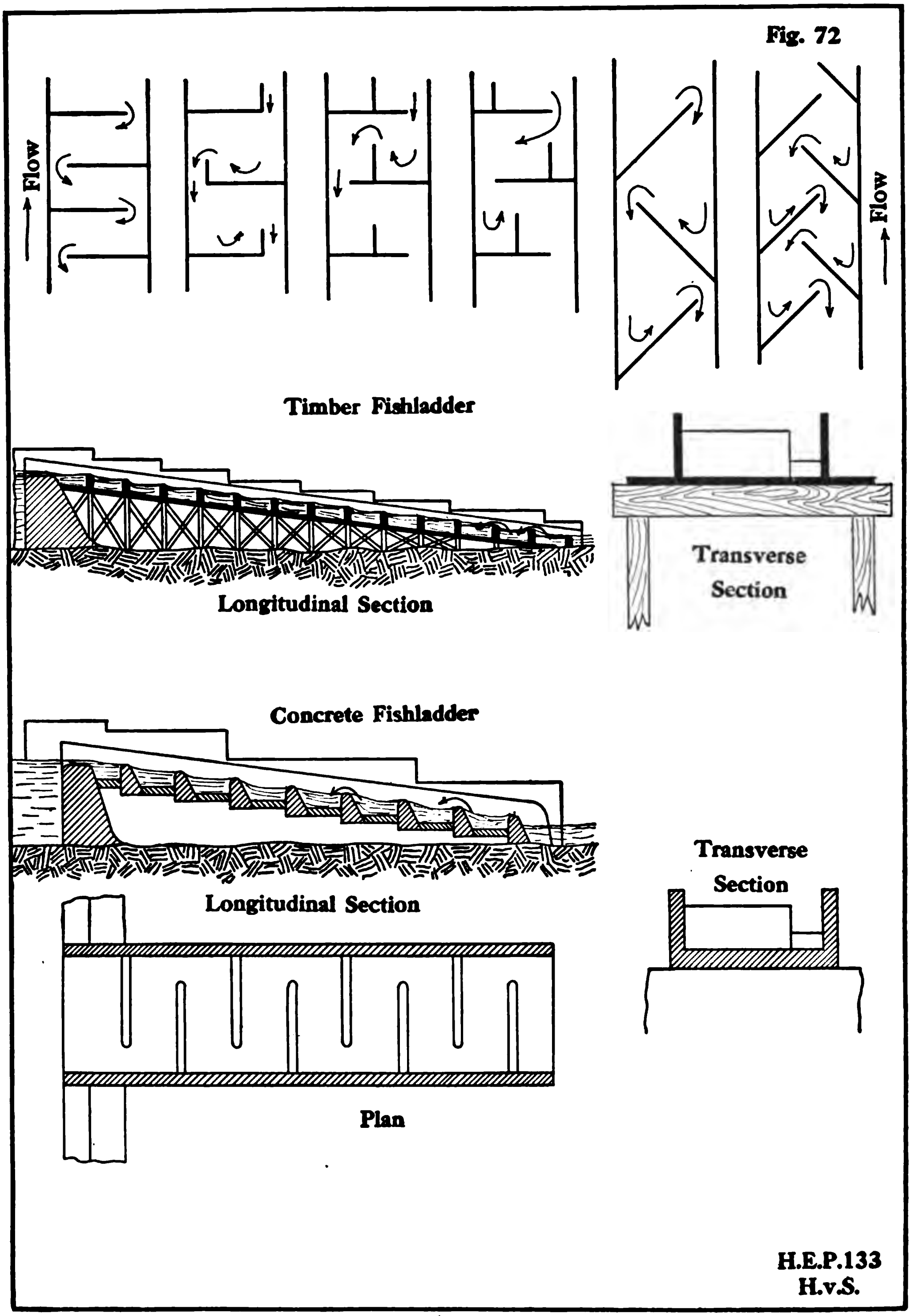
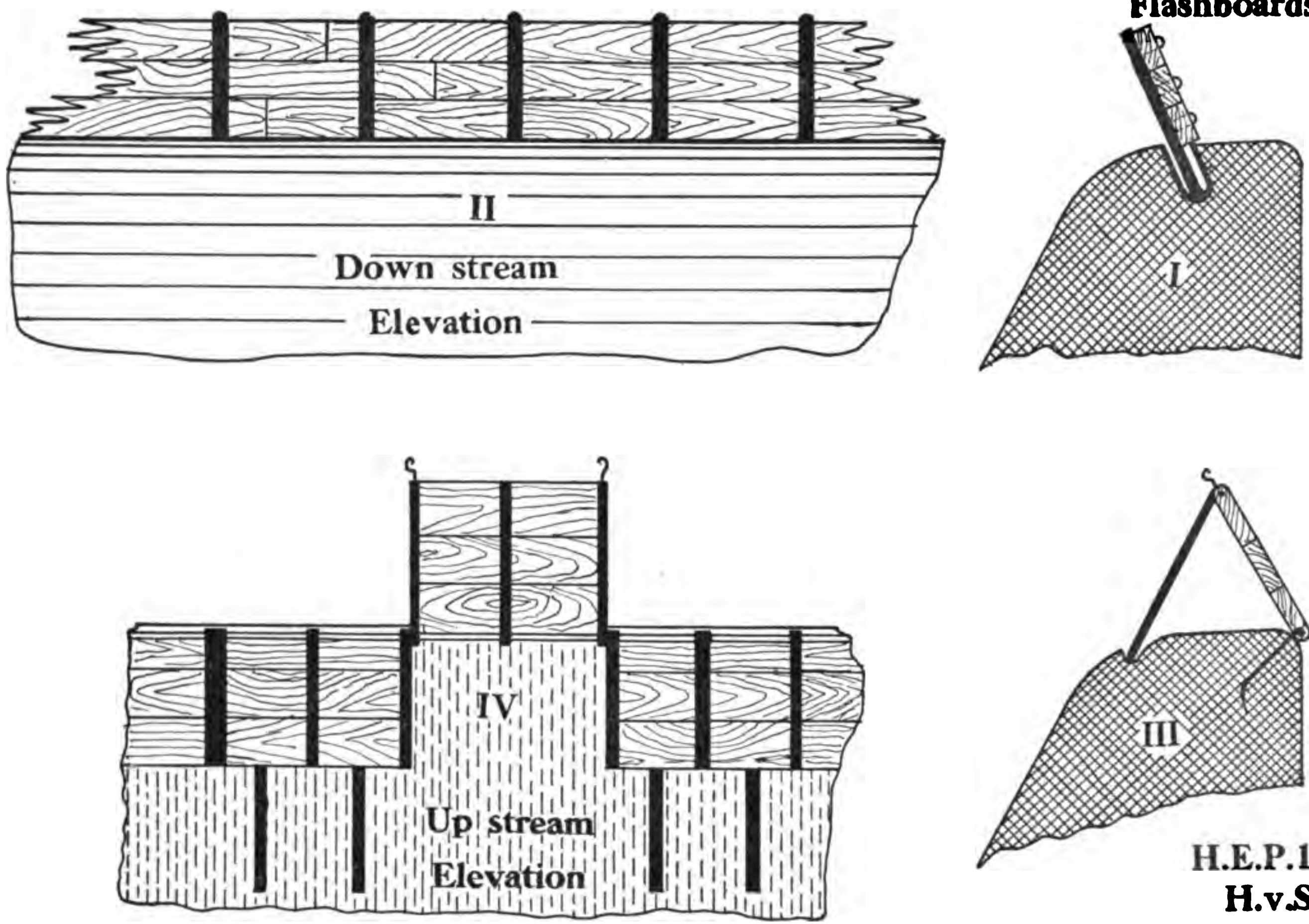


Fig. 72
Flow
Flow
Timber Fishladder
Longitudinal Section
Transverse
Section
Concrete Fishladder
Longitudinal Section
Transverse
Section
Plan
H.E.P.133
H.v.S.

Fig. 73
Flashboards



H.E.P.134
H.v.S.

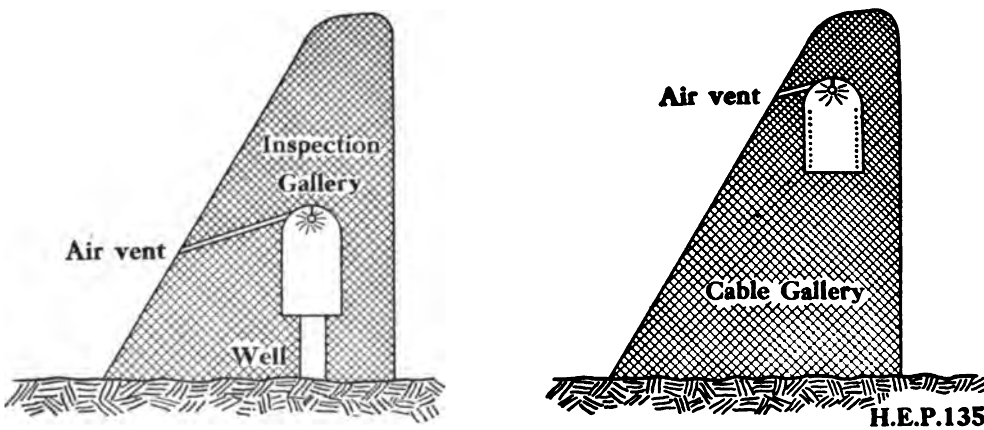
during a non-operating period. In this manner the upper pool *by pondage* is utilized as a storage reservoir, principally during the low-flow periods, by placing some movable addition upon the spillway crest, arresting and holding the natural flow in the upper pool, no water being allowed to overflow the spillway or pass through the turbines during a certain period, generally some portion of the night, and then using the natural flow plus the accumulated volume during the operating period. When this proceeding is feasible,—which is not always nor generally the case, because it interferes with and disturbs the natural conditions of the flow in the stream and thereby is likely to interfere with the rights and ownership, in and to the water, of others,—and when the operating period can be confined to ten hours, the power output of the continuous flow can in this manner be practically doubled.

Many different devices are employed for flashboard service, mechanical, automatic, and hand operated, but their selection should be guided by the conditions under which they are to be used as relating to seasons, periods, and frequency of their service. When they are to be employed during the winter, in northern latitudes, the ice conditions must be considered. Fig. 73 shows different types of flashboards. In sections I and II a *plank flashboard* is set on edge upon the spillway crest resting against strain pins, inclined downstreamward and placed into holes left in the spillway masonry, or between such strain pins, the latter being arranged in a double row and staggered; these planks may be so placed by being handled from a platform above or from flat-boats held in place above the spillway by guide-lines secured to shore points or to a ferry line crossing the stream above the spillway. This type of flashboards answers well during open seasons for short spillways; the strain pins are of one-inch wrought iron; it is advisable not to fix them permanently into the masonry, but to leave holes into which they are readily set; the planks should be two inches thick and from 8 to 12 inches wide, though the narrower size will be found preferable on account of the greater ease of handling them; and the planks have secured to them, on their sides, iron hooks so set that they can easily be grappled into. The planks must be of uniform thickness and edged so that they will match up to a water-tight wall in any position. It requires two men to set or remove these flashboards; they are inexpensive and can be quickly handled.

Sections III and IV show a *shutter flashboard* framed of two or three planks in lengths from 6 to 10 feet and swung from rod hinges on the

upstream side of the spillway. The shutter is bound by iron strapping and has secured to it two or more strut rods by which the shutter is supported in an upright or inclined position when erected, these rods falling into recesses or grooves left in the spillway crest for that purpose. They are operated by hand, being raised to the surface, strut rods thrown over on the downstream face, and the shutter then raised up where it will be held by the water pressure of the rising upper pool; or the shutters may be arranged to rest against stationary or removable strain pins as in the case of the plank flashboard.

Fig. 74



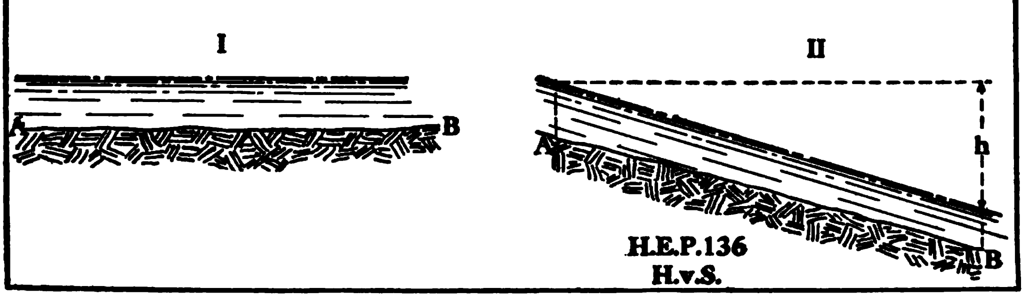
Wells and galleries are arranged in solid spillways for the purpose of inspection, means of communication and location of wire cables; as shown in Fig. 74. Access to these is had through the downstream side of the abutments and a casemate entrance arranged in the downstream slope of the embankment. They are formed of the least required dimensions; air-vents of two-inch galvanized iron pipe are placed 10 feet c. to c. from the gallery wall to the spillway apron face, with a slight downward drop to prevent water from passing through them.

Bridges, foot-walks, and *operation platforms,* for one purpose or another, of timber or steel construction, may be arranged on the spillway.

Article 75.—*Diversion works* comprise the structures conducting the water to the power station; they may be classified as canals, flumes, and pipe lines.

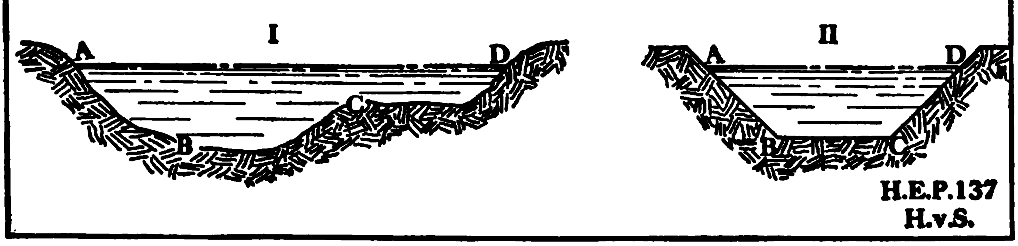
Canals serve to divert volumes exceeding 500 cubic second feet; their design is based upon the theory of *flow in open channels*. Fig. 75, section I, represents an open channel section the bed of which, AB, is in a horizontal plane, and, if this condition prevails from the source to

Fig. 75



the terminal, *the water does not flow;* in section II the bed AB is downwardly inclined and the water *flows* through it, its surface assuming a *slope* similar to that of the channel bed; the vertical difference of the horizontal plane of the water surface at A and B, or at any two points,

Fig. 76



is the *head h* between such points, which causes the flow and which is generally expressed in terms of slope $S = h \div d$, where d is the length of the channel AB and both are in like units, also called the *hydraulic gradient.*

Fig. 76, section I, represents the natural section of a stream, section II that of a constructed channel; ABCD is the *wet perimeter*, "P"; the cross sectional area A of the stream, divided by the developed length of

the wet perimeter, is the *hydraulic radius* "R." The flow in open channels depends upon the values of S, A, P and the degree of roughness "n" of P, and, from the relations of these, expressions have been developed, as the results of many observations and experiments, for the mean velocity of the flow, the basic form of which is:

$$\text{velocity} = \text{coefficient into the square root of } R \times S,$$

$$v = C\sqrt{RS},$$

in which C is the variable quantity which, according to the present acceptation as deduced by Ganguillet and Kutter from results found by M. Bazin, is expressed as

$$C = y \div [1 + (x \div \sqrt{R})],$$

$$y = a + (1 \div n) + (m \div s), \text{ and } x = [a + (m \div s)] \times n, \text{ in which}$$

1 is a constant = 1.811 ft.,
a is a constant = 41.6,
m is a constant = 0.00281, and
n is variable;

its values for the application to diversion canals are for

channels lined with dressed planks or smooth concrete	= 0.010
channels lined with rough planks or rough concrete	= 0.012
channels lined with smooth natural rock	= 0.017
channels lined with hard gravel, clay, and sand	= 0.025
channels lined with soft alluvial materials	= 0.03

Ex.—A = 1000 sq. ft., P = 128 ft., R = 7.8,

$$S = 0.0005, \quad n = 0.025$$
$$y = 41.6 + (1.811 \div 0.025) + (0.00281 \div 0.0005) = 119.64$$
$$x = [41.6 + (0.00281 \div 0.0005)] \times 0.0005 = 1.18$$
$$x \div \sqrt{R} = 0.42$$
$$C = 119.64 \div 1.42 = 84.3$$
$$V = 84.3\sqrt{7.8 \times 0.0005} = 5.26 \text{ sec. ft.}$$

Values for the factors "y" and "x" for different slopes and perimeter conditions are given in Tables 16 and 17.

TABLE 16.—$y = a + (1 \div n) + (m \div S)$.

S.	n = 0.010	n = 0.012	n = 0.017	n = 0.025
.0001	250.8	220.7	175.7	142.1
.0002	236.7	206.6	161.6	128.1
.0003	232.0	201.9	156.9	123.4
.0004	220.7	199.6	154.6	121.1
.0005	228.3	198.2	153.2	119.6
.0006	227.4	197.3	152.3	118.7
.0007	226.7	196.6	151.6	118.0
.0008	226.2	196.1	151.1	117.5
.0009	225.8	195.7	150.7	117.2
.001	225.5	195.4	150.4	116.8

TABLE 17.—$x = [a + (m \div S)] \times n$.

.0001	0.70	0.84	1.18	1.74
.0002	0.56	0.67	0.95	1.38
.0003	0.51	0.61	0.87	1.27
.0004	0.48	0.58	0.83	1.22
.0005	0.47	0.57	0.80	1.18
.0006	0.46	0.55	0.79	1.16
.0007	0.45	0.54	0.77	1.14
.0008	0.45	0.54	0.77	1.13
.0009	0.44	0.53	0.76	1.12
.001	0.44	0.53	0.75	1.11

Analyzing the flow formula and the influence of its variable factors upon the resultant velocity value, in its special application to the designing of diversion channels, the range of the coefficient of roughness can be limited to those conditions of lined channels by which permanency of prism is absolutely guaranteed, as only in extremely rare cases would a power canal present any other conditions; that means that the first three values of "n" fully cover the various perimeters to be met with in this subject, and that the value of "y" is between 150 and 250 and of "x" between 0.4 and 1.2, and within this scope the values of the other pertinent expressions in the flow formula, such as $x \div \sqrt{R}$ and of "C," are given in the following tables.

TABLE 18.—$[1 + (x \div \sqrt{R})]$.

R	x = 0.4	x = 0.5	x = 0.6	x = 0.7	x = 0.8	x = 0.9	x = 1
1	1.4	1.5	1.6	1.7	1.8	1.9	2
2	1.28	1.35	1.42	1.50	1.57	1.64	1.70
3	1.23	1.30	1.35	1.40	1.47	1.52	1.60
4	1.20	1.25	1.30	1.35	1.40	1.45	1.50
5	1.18	1.22	1.27	1.31	1.36	1.40	1.45
6	1.16	1.20	1.25	1.29	1.32	1.37	1.40
7	1.15	1.19	1.23	1.26	1.30	1.34	1.38
8	1.14	1.18	1.21	1.24	1.28	1.32	1.35
9	1.13	1.17	1.20	1.23	1.26	1.30	1.33
10	1.12	1.16	1.19	1.22	1.25	1.28	1.32

This completes the analysis of the factor "C" in the flow formula.

Example.—n = 0.012, S = 0.0003; the channel is to divert 3000 cub. sec. ft. at a maximum velocity of 5 ft.,

$$A = 600 \text{ square feet,}$$

the side slopes of the canal are to be 1.5 horizontal in one vertical,

$$P = 113 \text{ ft. and } R = A \div P, = 600 \div 113 = 5.3,$$

$$y \text{ from Table 16} = 201.90,$$

$$x \text{ from Table 17} = 0.61,$$

$$[1 + (x \div \sqrt{R})] \text{ Table 18} = 1.26, \text{ and therefore}$$

$$C = y \div [1 + (x \div \sqrt{R})] = 201.9 \div 1.26 = 160.$$

Values for "C" for above y, x, and R are given in Table 19.

TABLE 19.—$C = y \div [1 + (x \div \sqrt{R})]$.

y	1.1	1.2	1.3	1.4	1.5	1.6	1.7	1.8	1.9	2.0=$[1+(x\div\sqrt{R})]$
250.........	227	208	192	179	167	156	147	139	132	125
240.........	218	200	184	171	160	150	141	133	126	120
230.........	209	191	177	164	153	144	135	128	121	115
220.........	200	183	170	157	147	137	130	122	116	110
210.........	191	175	161	150	140	131	123	117	110	105
200.........	182	166	154	143	133	125	117	111	105	100
190.........	173	158	146	136	127	119	112	105	100	95
180.........	164	150	138	129	120	112	106	100	95	90
170.........	154	141	131	121	113	106	100	94	90	85
160.........	145	133	123	114	107	100	94	90	84	80
150.........	136	123	115	107	100	93	88	83	80	75

Continuing the solution of the previous example, "C" can be taken off Table 19 by interpolation and the velocity found from

$$v = C\sqrt{RS}, \ \sqrt{RS} = \sqrt{5.3 \times 0.0003} = 0.0398.$$

$$v = 160 \times 0.0398 = 6.368.$$

Velocity in a diversion canal should not be excessive; as a rule, it will be important to conserve the available head wherever practicable, and the design of the canal affords one of the important opportunities to

practise this economy. Only where the excavation of the canal prism presents a very costly undertaking, such as when it has to be located through a hard rock ledge, or in case the value of the necessary right of way is practically prohibitive and when the canal section therefore must be kept at a minimum, are high velocities excusable; five feet per second is a good limit to be adopted for the flow in a diversion canal, and within this limit the value of "v" is given for different slopes and R in the following Tables 20 to 24.

TABLE 20.—VELOCITY FOR S = 0.0001.

	n = 0.010			0.012		0.017	
R	C	$\sqrt{RS}$	v	C	v	C	v
1	147	0.01	1.47	120	1.20	81	0.81
2	168	0.014	2.35	138	1.93	96	1.34
3	178	0.017	3.02	149	2.53	104	1.77
4	186	0.02	3.92	155	3.15	111	2.22
5	191	0.022	4.20	160	3.52	116	2.55
6	195	0.024	4.68	164	3.93	119	2.85
7	198	0.026	5.34	167	4.34	122	3.17
8	201	0.028	....	170	4.76	124	3.47
9	203	0.03	....	172	5.16	126	3.78
10	205	0.03	....	174	5.40	128	3.96

TABLE 21.—VELOCITY FOR S = 0.0002.

	n = 0.010			0.012		0.017	
R	C	$\sqrt{RS}$	v	C	v	C	v
1	151	0.014	2.11	123	1.72	83	1.16
2	170	0.02	3.40	140	2.80	97	1.94
3	179	0.024	4.29	149	3.57	105	2.52
4	185	0.028	5.18	155	4.34	111	3.11
5	190	0.031	5.89	159	4.93	114	3.53
6	193	0.034	....	162	5.50	117	3.98
7	196	0.037	....	165	....	120	4.44
8	198	0.040	....	167	....	122	4.88
9	200	0.043	....	169	....	124	5.33
10	201	0.045	....	170	....	125	5.62

TABLE 22.—VELOCITY FOR S = 0.0003.

	n = 0.010			0.012		0.017	
R	C	$\sqrt{RS}$	v	C	v	C	v
1	152	0.017	2.58	124	2.10	84	1.42
2	171	0.024	4.10	140	3.36	97	2.32
3	179	0.030	5.37	152	4.56	105	3.15
4	185	0.034	....	156	5.30	111	3.77
5	189	0.038	....	159	....	114	4.33
6	192	0.043	....	162	....	117	5.03
7	195	0.046	....	164	....	119	5.47
8	197	0.049	....	166	....	121	....
9	198	0.053	....	168	....	123	....
10	200	0.055	....	169	....	124	....

TABLE 23.—VELOCITY FOR S = 0.0004.

	n = 0.010			0.012		0.017	
R	C	$\sqrt{RS}$	v	C	v	C	v
1	154	0.020	3.08	125	2.50	85	1.70
2	171	0.028	4.78	141	3.94	98	2.74
3	180	0.034	6.12	150	5.10	105	3.57
4	184	0.040	....	157	6.28	110	4.40
5	188	0.045	....	159	....	113	5.08
6	191	0.049	....	161	....	116	5.68
7	193	0.053	....	163	....	118	....
8	195	0.056	....	165	....	120	....
9	197	0.060	....	167	....	122	....
10	199	0.063	....	168	....	123	....

TABLE 24.—VELOCITY FOR S = 0.0005.

	n = 0.010			0.012		0.017	
R	C	$\sqrt{RS}$	v	C	v	C	v
1	154	0.024	3.68	125	3.00	85	2.04
2	175	0.031	5.30	141	4.37	98	3.04
3	179	0.039	....	150	5.85	105	4.09
4	184	0.045	....	156	....	110	4.95
5	188	0.050	....	158	....	113	5.65
6	191	0.055	....	161	....	116	....
7	193	0.059	....	163	....	118	....
8	196	0.063	....	165	....	120	....
9	197	0.067	....	167	....	122	....
10	199	0.070	....	168	....	123	....

By aid of the tabulated values in these nine tables all problems relating to the flow in a diversion canal or flume can be solved or checked; the frequent query, of what the slope would be in a canal of certain prism and a given velocity of flow, is thus solved.

Example.—To divert 800 cubic second feet at a velocity of approximately 5 feet per second in a rectangular sectioned canal 5 feet deep and 32 feet wide; what will be the slope?

$$A = 800 \div 5 = 160 \text{ sq. ft.}, R = 160 \div 42 = 3.8, n = 0.012,$$

and velocity to be about 5 feet per second.

From Table 22 in column of n = 0.012, and between values for R of 3 and 4, the desired velocity appears and the slope is 0.0003. It must be noted that the slope is expressed in the ratio per foot of length, a slope of 0.0001 = 1.2 inch per 1000 feet and 6.33 inches per mile.

Location, construction, and operating conditions are the main considerations for the designing of the diversion canal.

The location should be the economically shortest, which is determined by the cost of the right of way and of the construction, and is most readily proved by the method of elimination,—that is, by making paper locations based upon the survey and boring data and finding the excavation quantities and slope areas for a flow section of one-fifth of the maximum volume to be diverted, thus assuming a trial velocity of five feet. Deviations from a tangent alignment will generally prove justifiable to avoid side-hill cuts, buildings, road crossings, rock outcrops, swamps, and to secure uniformity of the prism, but curvature should be limited to 3°.

The slope in curved channels is greater than in straight; the excess is determined from Humphrey and Abbott's formula,

$$hc = v^2 \times 6\ d \div 536\ p,$$

where v is the mean velocity, d the total angle of the curve expressed in radians (1° = 0.01745), and p = 3.1415.

Example.—In a channel with v = 5 ft., on a 3° curve 900 ft. long, the curve slope $hc = 5^2 \times 6 \times 27 \times 0.01745 \div 1683$,

$$= 70.6725 \div 1683 = 0.42,$$

which must be added to the slope in a straight channel of the same length, or, if the general slope in this case is 0.00015, the total slope in this curve is 0.135 + 0.042 = 0.177 ft.

The construction considerations to be weighed for the purpose of deciding upon the location and design of the canal pertain to the excavation of the prism, the lining of the bed and slopes, and the revetting of the superbanks.

Excavation cost depends upon the character of the material, the quantities to be moved, and the disposition which may be made of the spoils; all these, together with the depth and width of the cut, will influence the methods which secure the most economical excavation. Rock is most cheaply removed in the dry; the principal operations involved in *excavating hard rock*, a classification which includes those formations which can be excavated in large masses only by the use of explosives, are drilling, blasting, loading, and disposing.

Rock drilling may be done by hand tools or by machine drills operated by steam or compressed air; only machine drilling will be here considered.

This operation requires a runner and helper, power being supplied from a central plant. The output varies with the depths of the holes, being from 50 to 60 linear feet for 10 and 20 feet depths per shift of 10 hours. To the operating cost must be added the power cost for 10 horse-power per drill, the drill repairs and drill sharpening; with present wages drilling costs from 10 to 15 cents per linear foot, depending on the hardness of the rock and the depth of the holes.

The breaking of the rock for the purpose of loading for removal requires from 1 to 1.5 pounds of 60 per cent. dynamite, according to its hardness, per cubic yard of output, and with present cost of explosives a charge of 15 cents per cubic yard should be made for blasting. The broken rock may be *loaded by hand*, or machinery, into carts, derrick or cable-way buckets, or dump cars, and the cost varies in accordance with the method employed; the output of hand loading averages 10 cubic yards per man per 10-hour shift, by steam shovel it will be between 40 and 60 cubic yards per hour. *Disposal cost* depends entirely upon the length of *haul*. With present wages rock excavation will cost from 75 cents to $1.50 per cubic yard.

Earth is removed more cheaply by dredging than by dry excavation; the cost of dredging will be generally from 15 to 30 cents, depending upon the hardness of the material and the distance of the disposal. *Dry earth excavation* may be done by hand tools, horse and power scrapers, or by steam shovels, the operations consisting of loosening, loading, and disposing; the output and cost depend upon the methods employed, the character of the material, depth of cut, and distance of haul. With present wages the cost will be from 25 to 50 cents per cubic yard. Comparatively, the cost of rock excavation is about three times that of earth, and dredging about 0.6 of earth excavation. *The quantity of earth excavation* is about 1.2 of the flow area up to the water level, and approximately one-third of that area for every foot vertical above water surface.

Example.—For a canal which is to divert 1000 cub. sec. ft. at a velocity of 5 ft. per sec. the quantity to be excavated to the water level is $= 200 \times 1.2 = 240$ cub. ft. per linear foot of canal, and to a height of 5 ft. above the water level it is $= 240 + (200 \times 1.66) = 570$ cub. ft. or about 22 cub. yds.

For side-hill locations the quantities must be determined from cross-sections taken at intervals of 10 feet.

The bed and canal sides (Fig. 77, 1) in rock location should be finished as smooth as practicable, in order to realize the highest flow efficiency, which will generally require hand-tool finishing of the bed, while the sides should be cut out by channelers, which with present wages costs from 15 to 18 cents per square foot surface, depending upon the hardness of the rock and the depth of the channel cut. In alluvial locations the bed and canal slopes should be covered with a lining, in order to guarantee the permanency of the prism and reduce the roughness to a minimum and thereby secure the best flow efficiency. *Canal lining* may be of timber or concrete-steel. Timber lining (Fig. 77, 2, 3, and 4) consists of 3-inch planking laid longitudinally upon 12 × 12 inch timber sills spaced 8 ft. c. to c. and imbedded in the bed material, being secured in place by connection to bearing piles from 8 to 16 ft. long, or by iron rods of the type described for the lining of a reservoir embankment slope in Article 73. A lining of concrete-steel (Fig. 77, 5, 6, and 7) may be laid upon transverse timber sills, as above, or upon concrete sills 8 inches square, which need no further support unless the underlying material is mud, when piles must be driven. The concrete lining is 6 inches thick for sills 8 ft. centres, and is connected with the sills by one-inch dowels set 4 ft. c to c. The concrete lining contains reinforcing steel. The canal slopes should be no steeper than 1.5 horizontal in one vertical; they may be lined in the manner described for the bed, being structurally continuous of the bed lining and terminating, when of timber, one foot below the normal flow level in a *berme* (shown in Fig. 77, 2), and, when of concrete, 2 feet above such elevation (as shown in Fig. 77, 5).

The superbanks of the canal should be sloped at two horizontal in one vertical and *paved*.

Views 7 and 8 show a rock canal with channelled sides and paved superbanks, Views 9, 10, 11 an earth canal with timber lining.

From these data estimates for various locations and prisms can readily be compiled, and their comparative cost will point to the most economical.

Appurtenant structures to diversion canals are those required for the safeguarding of the superbanks against erosion from surface run-off, interception of unavoidable lateral stream sources, the devices controlling the flow in the canal, and means of overhead crossings.

Ordinary run-off from rainfall is best intercepted by a longitudinal paved drain located at the top of the superbanks, from which laterals

View 7

Canal in Rock
Excavating and Channelling

View 8

Canal in Rock
with Channelled Sides
Completed

View 9

Canal in Earth,
Timber lined
Laying the Sills

View 10

Canal in Earth,
Timber lined
Planking

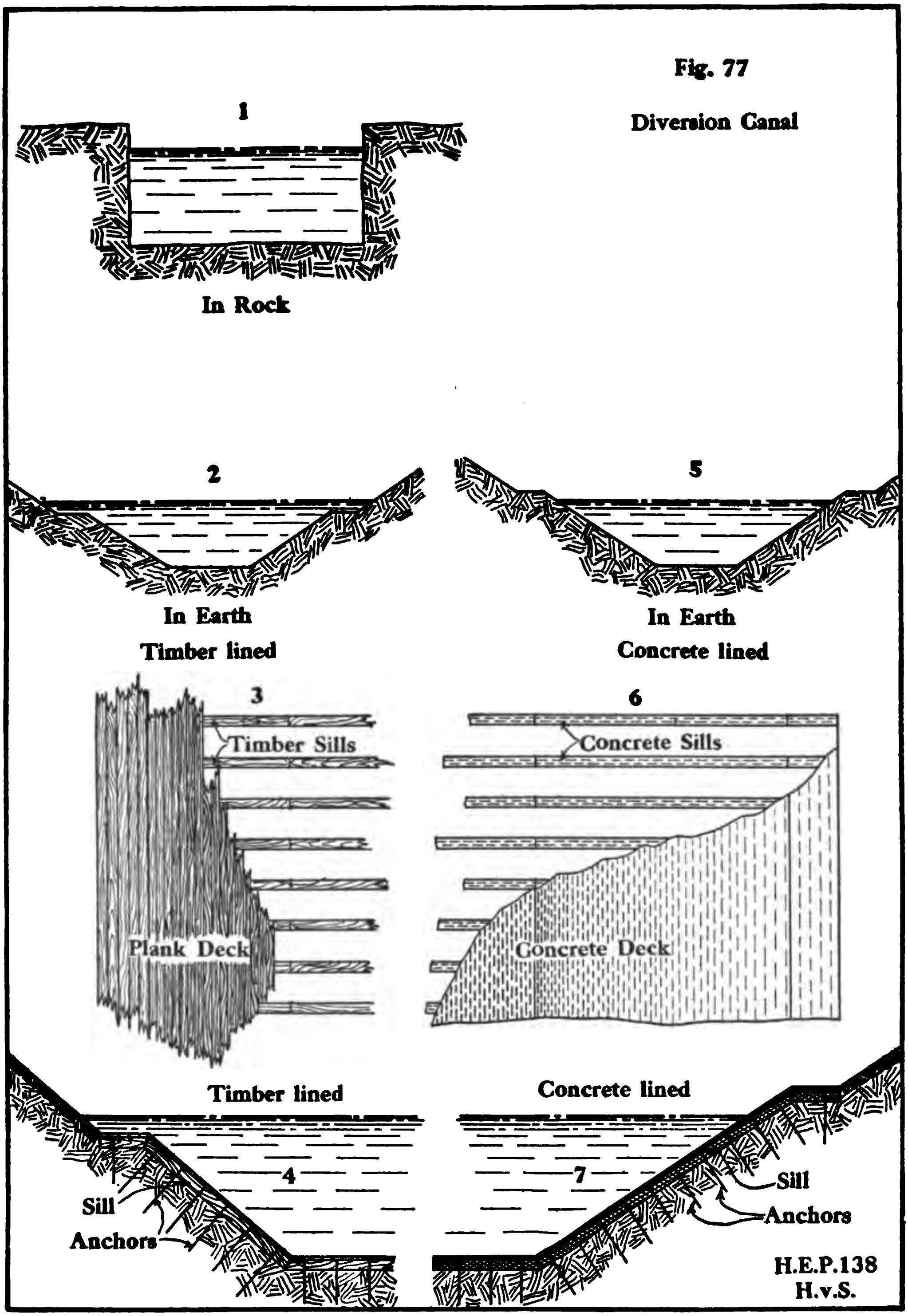


Fig. 77
Diversion Canal
1
In Rock
2
In Earth
Timber lined
5
In Earth
Concrete lined
3
Timber Sills
Plank Deck
Timber lined
6
Concrete Sills
Concrete Deck
Concrete lined
4
Sill
Anchors
7
Sill
Anchors
H.E.P.138
H.v.S.

of similar type are led down the slopes at intervals, depending upon the volume likely to accumulate, of from 100 to 300 feet. When the canal traverses *ravines* which form stream sources after heavy rainfall, provision must be made to pass such flow under the canal by means of concrete culverts, unless the ravine can be made part of the diversion canal by securely cutting off its terminal and connecting the canal banks with those of the ravine; when the volume which may pass down the ravine is likely to exceed 0.1 of the canal flow area, an *overflow* must be constructed in the canal bank to pass the excess. Generally speaking, it will always prove the safer practice to care for such exterior flow sources by passage under the canal, the structure being of the concrete culvert type *and of ample dimensions.*

The canal entrance is guarded by *headgates,* which should afford complete and ready control of all flow into the canal; they may be of a variety of types as suggested by the volume of the flow and the operating requirements. *Stop-logs,* which were described in connection with the open spillway in Article 68, are a simple, effective, and inexpensive device for headgate service, being placed and operated in the same manner as when employed for the closing of overflow sluices. *Needles,* also described in Article 68, may be used, or lift-gates of timber or steel framing. A *foot-bridge* is arranged at the headgate crossing and serves as an operating platform. Some headgate designs are shown in Fig. 78 and in Views 12, 13, and 14.

An intake to the canal is frequently arranged above the headgates for the purpose of creating a readily accessible pool in which floatage can be intercepted and prevented from passing into the canal; it may also be the means of securing a more complete diversion of the low flow into the canal.

A forebay is likewise arranged at the terminal of the canal, being simply a gradual enlargement in which the velocity of the flow is reduced before the water enters the power house; its dimensions are decided by those of the power house and by the character of the turbine installation.

A waste weir should be arranged near the end of the canal, being practically a short open spillway with one or two overflow sluices; floatage, ice, and surplus flow may be passed over it.

Bridges are often required across a canal for operating purposes and to accommodate public traffic.

Diversion is secured by *flumes* when the volume is less than 500 sec. ft. Flumes are rectangular or elliptical timber conduits of designs and construction shown in Fig. 79; they are supported on timber trestles

View 11

Canal in Earth, Timber lined Completed

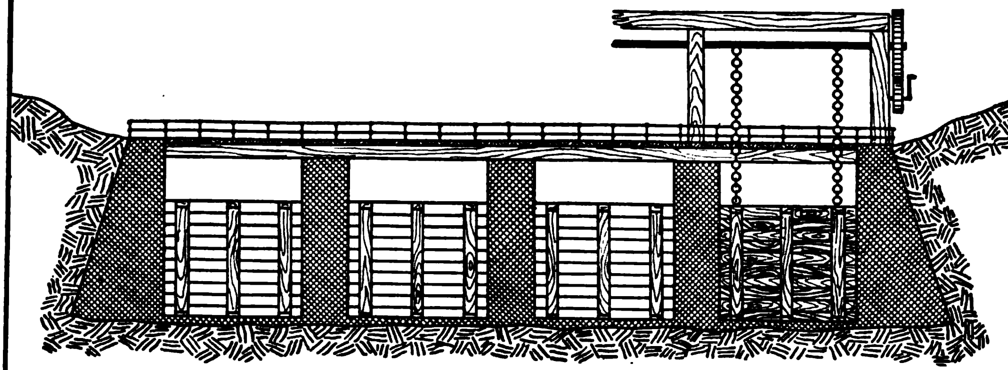
View 12

Canal Head Gates Stony Sluices

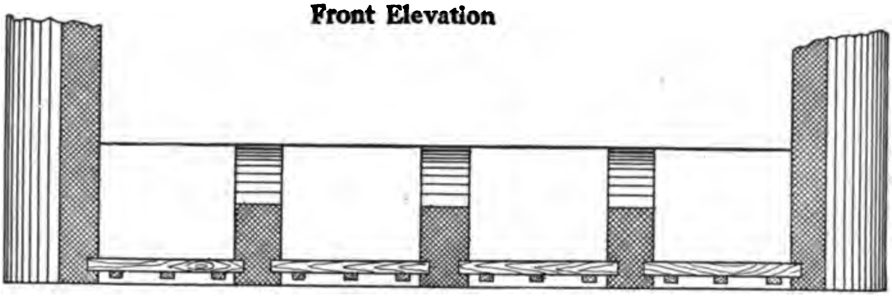

View 13
Canal Head Gate
H.E.P.146
H.v.S.

View 14.

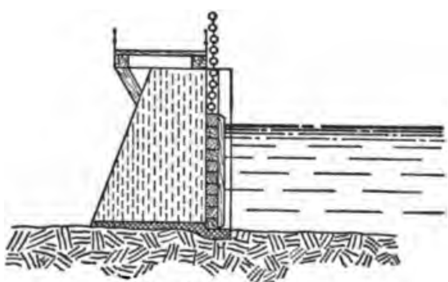

Timber Headgates
H.E.P.147
H. v. S.



Front Elevation



Ground Plan



Section

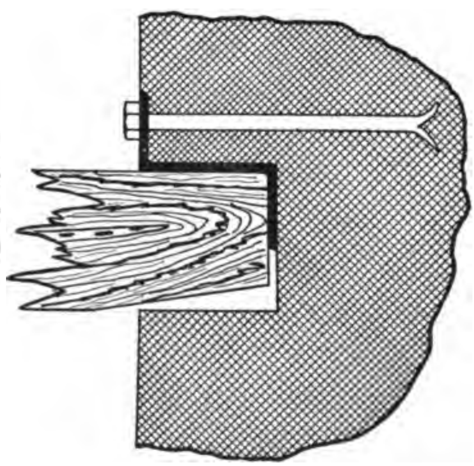


Fig. 78
Canal Head Gate

Detail of
Gate Seat

H.E.P.144
H.v.S.

or masonry piers or are placed on timber sills resting upon the surface of the ground. They should be water-tight, and their section is best confined to that required for the passage of the volume of water to be diverted, so that all parts of the conduit remain in a constantly saturated condition. The top should be covered with removable planks to protect the water surface against the sun and cold. The flow in flumes follows the laws found for open channels. The flume entrance is guarded by gates of stop-log, needles, or lift type.

Diversion in Pipes.—When the volume to be diverted is less than 500 sec. ft., or when the construction cost of a canal or flume is abnormally high, diversion in pipes may prove more economical; in high-head developments the final passage of the water to the turbines is always through pipes.

The flow of water in pipes is based upon the same general theory as that developed for flow in open channels, in accordance with the fundamental formula of $v = C\sqrt{RS}$, which has been analyzed in Article 75. The sizes of pipes for diversion service will generally be between 2 and 8 feet in diameter, the velocities from 2 to 6 feet per second, and the slopes from 0.001 to 0.0001.

TABLE 25.—VALUES WITHIN THE ABOVE LIMITS OF A, R, AND $\sqrt{R}$.

Diameter, inches.	Area, sq. ft.	Hydr. radius, R.	$\sqrt{R}$.
24	3.124	0.500	0.707
30	4.909	0.625	0.791
36	7.067	0.750	0.866
42	9.621	0.875	0.935
48	12.57	1.0	1.0
54	15.90	1.125	1.061
60	19.64	1.25	1.118
66	22.76	1.375	1.173
72	28.72	1.50	1.225
78	33.18	1.625	1.275
84	38.48	1.75	1.323
90	44.18	1.875	1.370
96	50.26	2.0	1.414

TABLE 26.—DISCHARGE VOLUME IN CUBIC SECOND FEET FOR VARIOUS SIZE PIPES AND VELOCITIES.

Diameter, inches.	2	2.5	3	3.5	4	4.5	5	5.5	6 ft. p. sec.
24	6.3	7.8	9.4	11.0	12.5	14.1	15.7	17.3	18.8
30	9.8	12.3	14.7	17.2	19.6	22.1	24.5	27.0	29.4
36	14.1	17.6	21.2	24.7	28.3	31.7	35.3	38.8	42.3
42	19.2	24.0	28.8	33.6	38.4	43.2	48.0	52.8	57.6
48	25.1	31.4	37.7	44.0	50.2	56.5	63.8	69.1	75.4
54	31.8	39.7	47.7	55.6	63.6	71.5	79.5	87.4	95.4

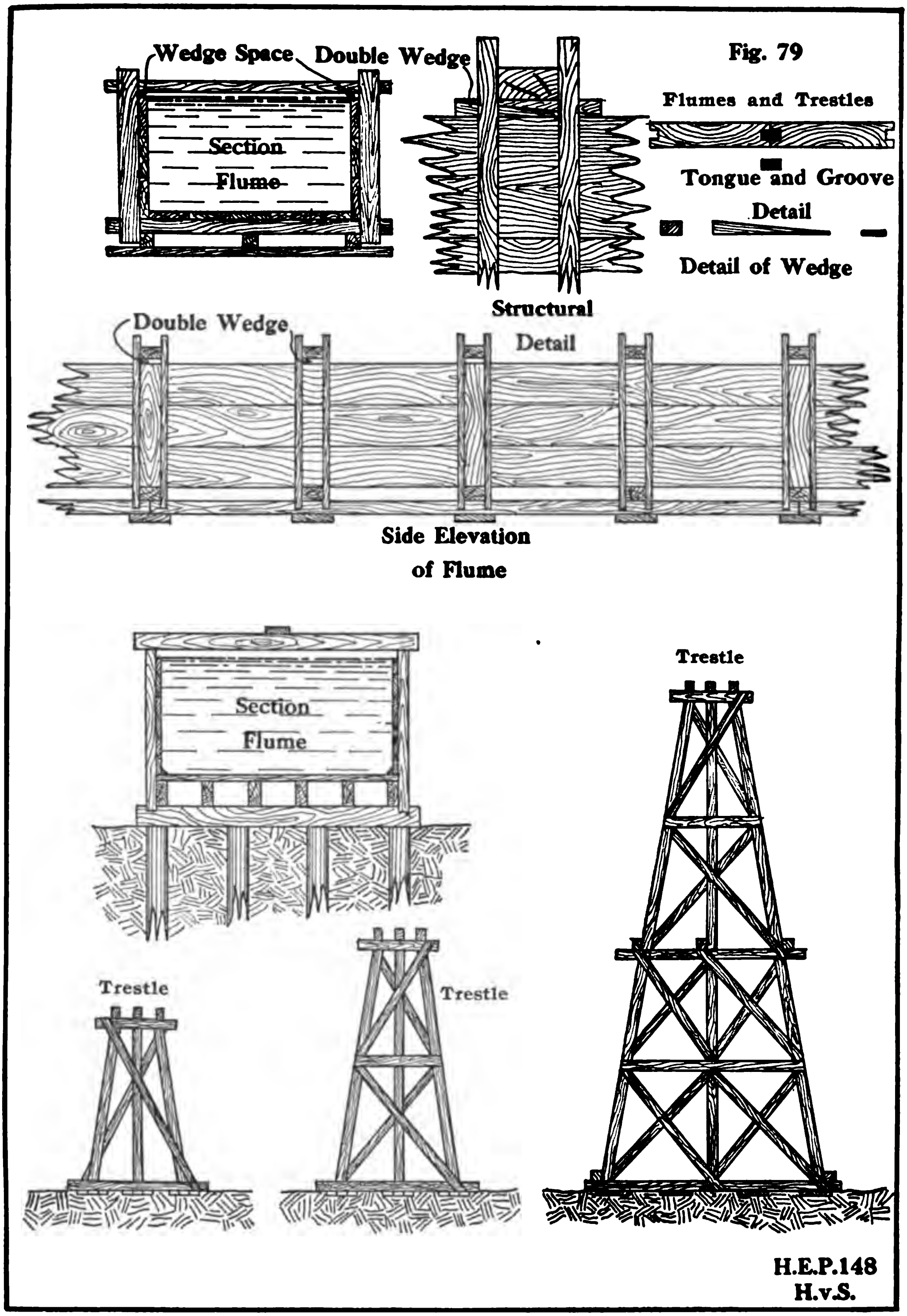


Fig. 79
Flumes and Trestles

TABLE 26.—DISCHARGE VOLUME IN CUBIC SECOND FEET FOR VARIOUS SIZE PIPES AND VELOCITIES.—*Continued.*

Diameter, inches.	2	2.5	3	3.5	4	4.5	5	5.5	6 ft. p. sec.
60....................	39.3	49.1	58.9	68.7	78.5	88.4	98.2	108.2	117.8
66....................	47.5	59.4	71.3	83.2	95.0	106.9	118.8	130.7	142.5
72....................	56.5	70.7	84.8	98.9	113.5	127.6	141.8	156.0	170.2
78....................	66.4	82.9	99.5	116.1	132.7	149.3	165.9	182.5	199.1
84....................	76.9	96.2	115.4	134.7	153.9	173.2	192.4	211.6	230.9
90....................	88.3	110.4	132.5	154.6	176.7	198.8	220.9	243.0	265.1
96....................	100.5	125.6	150.8	175.9	201.0	226.2	251.3	276.4	301.5

The value of "C" has been established by observations and experiments of flow in pipes, and is accepted as,

for smooth concrete pipes of ten years' service, C = 120,
for wooden stave pipes of ten years' service, C = 115,
for steel plate, riveted pipes of ten years' service, C = 110.

Tables 27, 28, and 29 give velocities for values of R, represented by diameter of pipe, and of S, for the above three expressions for C.

TABLE 27.—VELOCITIES WHEN C = 110.

Diameter, inches.	.0001	.0002	.0003	.0004	.0005	.0006	.0007	.0008	.0009	.001 = S.
24......................	0.78	1.08	1.35	1.56	1.74	1.91	2.06	2.20	2.34	2.46
30......................	0.89	1.22	1.54	1.78	1.98	2.18	2.35	2.51	2.67	2.80
36......................	0.95	1.33	1.64	1.90	2.43	2.32	2.50	2.69	2.85	3.00
42......................	1.02	1.44	1.77	2.04	2.28	2.50	2.72	2.88	3.06	3.23
48......................	1.10	1.56	1.01	2.20	2.46	2.70	2.93	3.11	3.30	3.48
54......................	1.16	1.65	2.01	2.32	2.59	2.83	3.09	3.28	3.48	3.67
60......................	1.23	1.74	2.13	2.46	2.75	3.01	3.28	3.47	3.69	3.89
66......................	1.20	1.82	2.24	2.58	2.88	3.16	3.44	3.64	3.87	4.08
72......................	1.34	1.90	2.33	2.64	3.00	3.27	3.60	3.78	4.02	4.25
78......................	1.40	1.98	2.44	2.80	3.14	3.43	3.76	3.96	4.20	4.44
84......................	1.45	2.05	2.53	2.90	3.23	3.55	3.84	4.10	4.35	4.57
90......................	1.50	2.12	2.60	3.00	3.35	3.67	4.01	4.24	4.50	4.74
96......................	1.55	2.19	2.66	3.12	3.46	3.79	4.16	4.38	4.65	4.91

TABLE 28.—VELOCITY WITH C = 115.

Diameter.	.0001	.0002	.0003	.0004	.0005	.0006	.0007	.0008	.0009	.001
24......................	0.81	1.14	1.40	1.62	1.81	1.98	2.13	2.29	2.43	2.56
30......................	0.91	1.28	1.57	1.82	2.03	2.22	2.40	2.57	2.73	2.87
36......................	1.00	1.41	1.73	2.00	2.24	2.44	2.65	2.83	3.00	3.16
42......................	1.07	1.51	1.85	2.14	2.40	2.61	2.85	3.06	3.21	3.40
48......................	1.15	1.63	1.99	2.30	2.57	2.80	3.06	3.29	3.45	3.65
54......................	1.22	1.73	2.11	2.44	2.73	2.96	3.25	3.50	3.66	3.87
60......................	1.28	1.81	2.19	2.56	2.86	3.12	3.42	3.65	3.84	4.06
66......................	1.35	1.91	2.32	2.70	3.02	3.30	3.59	3.85	4.05	4.27
72......................	1.41	1.99	2.43	2.82	3.15	3.44	3.75	4.05	4.24	4.46
78......................	1.46	2.07	2.51	2.92	3.28	3.57	3.90	4.20	4.38	4.63
84......................	1.52	2.15	2.61	3.00	3.40	3.71	4.07	4.40	4.66	4.82
90......................	1.54	2.23	2.72	3.14	3.50	3.83	4.17	4.50	4.71	4.95
96......................	1.62	2.29	2.80	3.24	3.64	3.94	4.35	4.66	4.86	5.15

TABLE 29.—VELOCITY WITH C = 120.

Diameter.	.0001	.0002	.0003	.0004	.0005	.0006	.0007	.0008	.0009	.001
24	0.93	1.30	1.61	1.86	2.07	2.28	2.45	2.63	2.79	2.94
30	1.07	1.52	1.85	2.14	2.40	2.62	2.83	3.02	3.21	3.38
36	1.14	1.61	1.95	2.28	2.55	2.79	3.00	3.22	3.42	3.60
42	1.22	1.73	2.10	2.40	2.73	2.98	3.26	3.43	3.66	3.82
48	1.32	1.87	2.25	2.64	2.95	3.22	3.50	3.74	3.96	4.17
54	1.39	1.97	2.40	2.78	3.10	3.40	3.69	3.95	4.17	4.39
60	1.48	2.10	2.56	2.96	3.32	3.62	3.93	4.21	4.44	4.68
66	1.55	2.19	2.68	3.10	3.52	3.80	4.10	4.42	4.65	4.90
72	1.60	2.26	2.76	3.20	3.58	3.92	4.25	4.58	4.80	5.06
78	1.68	2.38	2.90	3.36	3.76	4.11	4.45	4.80	5.04	5.31
84	1.74	2.46	3.00	3.48	3.90	4.26	4.62	4.98	5.22	5.50
90	1.80	2.55	3.10	3.60	4.03	4.41	4.76	5.12	5.40	5.69
96	1.86	2.65	3.22	3.72	4.16	4.55	4.94	5.30	5.58	5.88

Location, kind and size of pipe are to be determined when planning a pipe line; velocity and slope are fixed by the volume to be diverted and the diameter of the pipe.

The location of a diversion pipe line is to be chosen to secure the economically shortest in distance while insuring the preservation of the pipe. Any kind of pipe wears longer if lying entirely above the surface of the ground with air freely circulating all around it; it can then be readily examined, repainted, and kept in proper condition; therefore, if avoidable, pipe should not be wholly or partially buried. Thorough underdraining should be provided, so that no water will accumulate beneath it at any point.

In Fig 80 the straight line ln, connecting the upper and lower pool surfaces, is the *hydraulic gradient*; no part of the pipe line should rise more than 25 feet above this gradient, but may otherwise vary in elevations and grades. The pipe must be anchored at intervals of 50 feet to concrete benches, shown in Fig. 80, 2, their dimensions depending upon the character of the material in which they must be placed and size and slope of pipe; the pipe proper is connected to the supporting structures by steel rods fastened to steel collars around the pipe; the rods should have turnbuckles to compensate for temperatures, that is extreme heat or cold; this device is shown in Fig. 80, 3. By the aid of Tables 27, 28, and 29 all problems of flow, discharge, and slope can be readily solved.

Example.—Required the diversion of 80 cubic second feet, total head 400 ft. Only steel plate pipe is available under such a head. $C = 110$.

From Table 26 find the size and velocity to yield the required discharge: in a 54-inch pipe with a velocity of 5 ft. discharge is 79.5 cub. sec. ft., in a 60-inch pipe with a velocity of 4 ft. discharge is 78.5 cub. sec. ft., in a 66-inch pipe with a velocity of 3.5 ft. discharge is 83.2 cub. sec. ft., and from Table 37 find S for 66-inch pipe and 3.5 ft. velocity = .0003. If a smaller size pipe is preferred, R is found from Table 25, say for

$$54\text{-inch pipe } \sqrt{R} = 1.061, \text{ then from } V = C\sqrt{RS}, \text{ and}$$
$$V \div C\sqrt{S} = \sqrt{S}, \text{ or } 5 \div 110 \times 1.061 = \sqrt{S}$$
$$(5 \div 116.710)^2 = \sqrt{S}, \text{ or } S = 0.0019.$$

The question of recommendable size of pipe is to be determined from balancing the excess pipe cost and the net earning value of the head represented by the difference in slope between the two sizes of pipe compared.

Example.—For the requirements of the last example, the line is 3500 feet long; 66-inch pipe costs $2.00 per linear foot more than the 54-inch pipe, the excess cost of the larger size pipe is therefore $7000.00; the difference in head will be 0.0019 — 0.00073 or 0.00127 × 3500, being 4.09 feet, which, with a flow of 80 cub. sec. ft., represents about 26.2 electrical horse-power (consult Diagram 3). If the net value of the current is $15.00 per horse-power per year, the amounts to be balanced against each other are

interest at 5 per cent. on excess pipe cost of $7000.00 = $350.00,
and net receipt from power represented by 4.09 feet h = 393.00;

in other words, the value of the head saved is greater than the interest on the investment and therefore the larger size pipe is recommendable.

The difference of first cost of the three types of pipes is small; the concrete-steel pipe is imperishable and calls for no maintenance expenditures; wooden stave pipe wears as well as does steel plate pipe, provided it is kept filled with water at a uniform pressure at all times; steel pipe requires recoating every third year.

The flow in pipes *is controlled* by valve gates of different types, which are placed at the intake, which latter must also be guarded by a *trash rack*, consisting of flat iron bars set in an iron frame, half inch centres, and secured to the pipe intake. Such an arrangement is shown in Fig. 81.

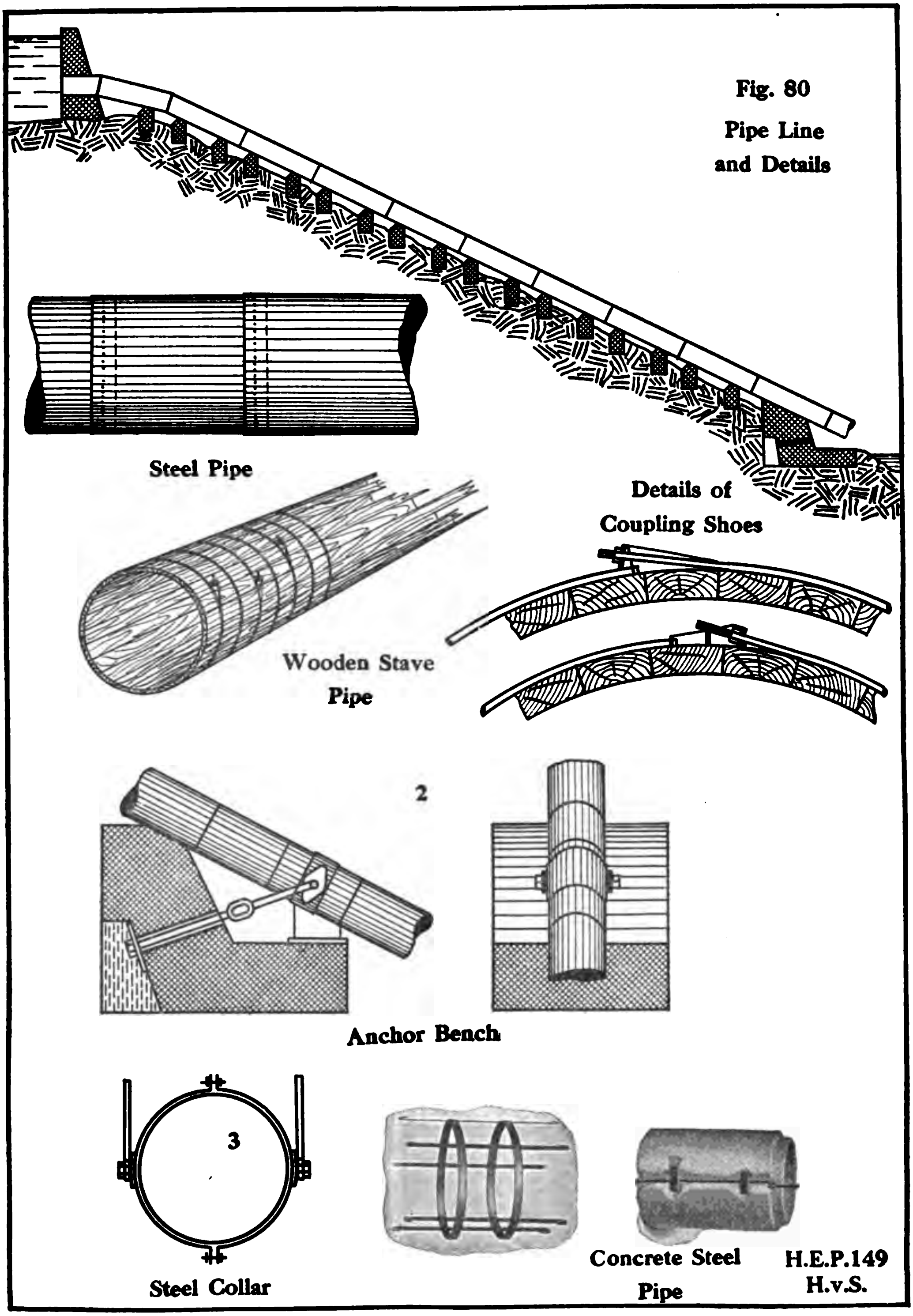


Fig. 80
Pipe Line
and Details

ARTICLE 76.—The diversion terminates at the *power house*. *The location* of this structure is determined as fully discussed in connection with the topic of development programmes in Article 48. Its type depends primarily upon the head. For low-head developments it will generally be located at the spillway or closely below it; with a medium head, from 30 to 60 feet, the power house may be at the spillway or at

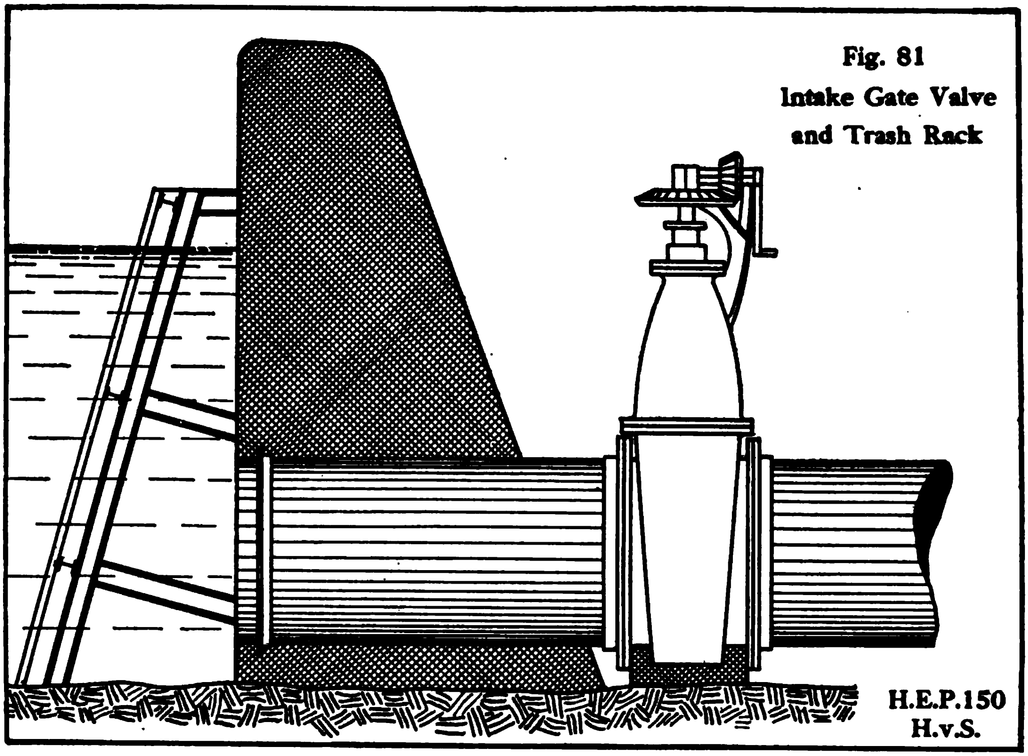


Fig. 81
Intake Gate Valve
and Trash Rack

the end of the diversion works; while for high-head developments it will always be at the terminal of a pressure line. Generally speaking, the location should be selected with a view to secure a good foundation, unobstructed efflux of the water,—that is, unimpeded by the overfall from the spillway, islands, rocks, or shoals,—convenient access to the building for conveyance of the equipment, protection against flood rise, ice, and logs, and, if practicable, the power house should be on the side of the river where the transmission line leads out.

The foundation considerations are the same as those described for the spillway, as the power house, in a sense, fulfils the functions of a

dam; the arrangement and construction of the power-house foundation are, therefore, similar to those outlined in Article 56 and the approximate quantities of material as given in Diagram 12. Here, as in the spillway case, the *cutting-off* of substrata, by which water might pass under the foundation, is of vital importance; in fact the power-house foundation in an alluvial location is best entirely surrounded by a substantial cut-off wall, the foundation proper overhanging the cut-off at least three feet. The sliding of the structure, under the pressure from the upper level water, must be likewise considered and guarded against, and an apron should be placed along the entire downstream side, being from 8 to 16 feet wide according to the depth of the water in the tail-race.

For plans of power-house foundation reference may be made to Plan 18, illustrating those for spillways. View 15 shows the foundation of a power house in an alluvial location, consisting of bearing piles, a timber grillage secured to them, and concrete placed between grillage frame and upon it. This was completely enveloped by a cut-off consisting of sheet piling and concrete wall.

The substructure of the power house, Fig. 82, elevates the station above the lower pool, supports it, and forms the pits into which the water from the turbines is discharged and by which it passes into the tail-race or directly into the lower pool. Power-house substructures are of practically one type, rectangular in plan, the length depending upon the number of power units, from 12 to 18 feet for each, according to the size of turbines, and the width conforming to that of the superstructure. The substructure consists of the forebay wall, Fig. 82, 1, which extends along the upstream side and must resist earth or hydrostatic pressures and supports the upper portion of the station, the end walls, Fig. 82, 2, also resisting lateral pressures and supporting the station building, the pit walls by which the different chambers are separated from each other and which must be designed for lateral pressure on one side in the event of the adjacent pit being unwatered; the pit floors and the pit roof complete the substructure. The walls may be of masonry, monolithic or block concrete, the floor and roof of concrete or concrete-steel. The substructure is therefore enclosed by walls except on the downstream ends, Fig. 82, 4, and these are arranged for the reception of gates or stop logs to enable the emptying of any of the pits. The height of the substructure is regulated by the water level in the lower pool, but the depth of water in the pits at the lowest level should not be less than five feet, as there

should be a four-feet water cushion below the discharge ends of the draft tubes. There is no objection that the highest lower pool water should stand at the under side of the pit roof.

View 16 shows such a power-house substructure as here described, and which may be termed a standard type; it is placed upon the foundation shown in View 15, this being the upstream elevation, while View 17

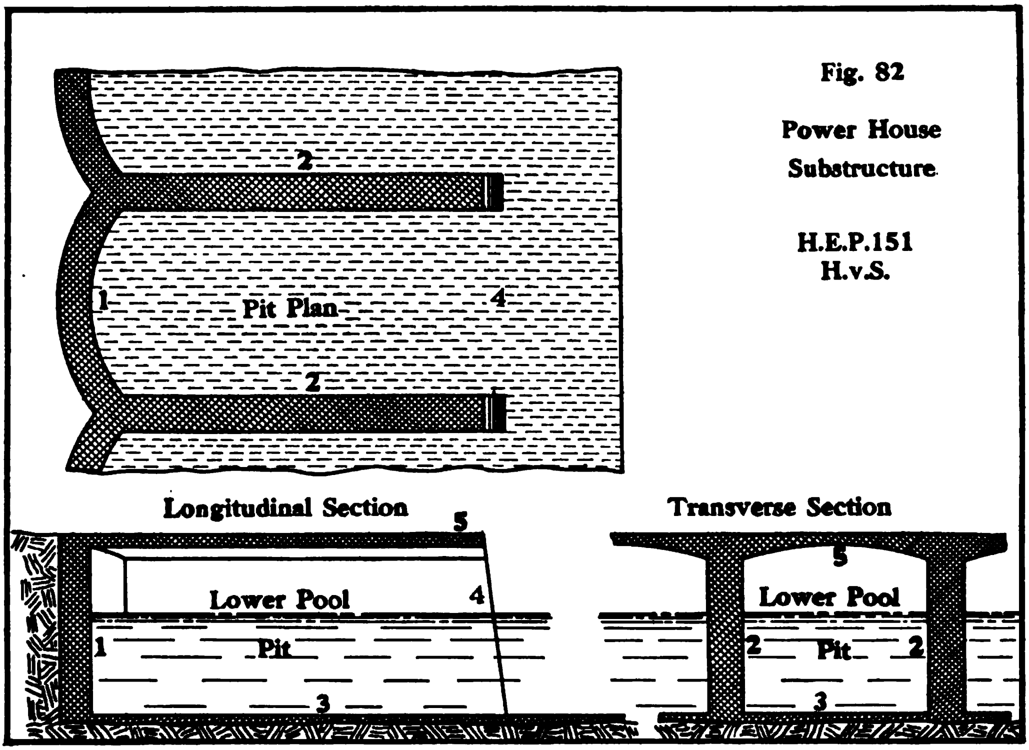


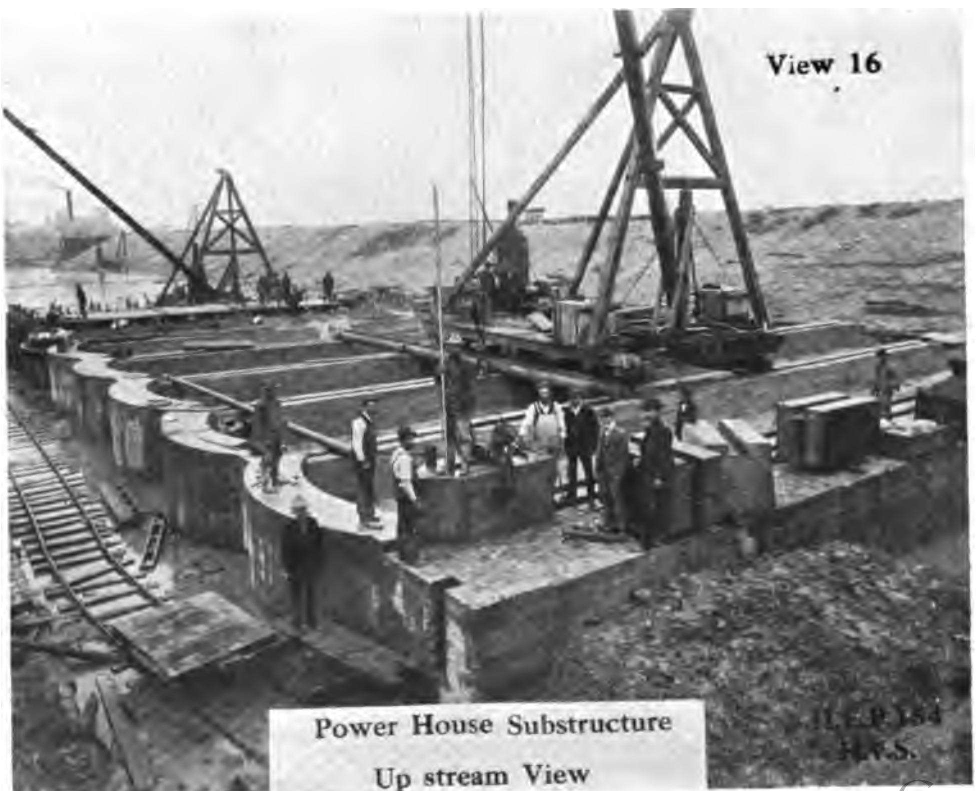
Fig. 82
Power House Substructure
H.E.P.151
H.v.S.

is of the downstream side. In this case the walls are all constructed of concrete blocks three feet thick, tongued and grooved all around; the floor and roof are of concrete-steel.

The superstructure houses the power equipment, and, depending upon the head, may be arranged for drowned or dry turbine installation. In the first case, Fig. 83, the turbines are placed in isolated bays into which the water enters freely with the upper level elevation, and passes through the turbines into the pit below; the pit roof forms the turbine-bay floor. The width and length of the turbine bays, Fig. 83, 1, are regulated by the installation space required for the turbines, gener-

View 15

Power House Foundation



Power House Substructure
Up stream View

View 17

Power House Substructure
Down stream Elevation

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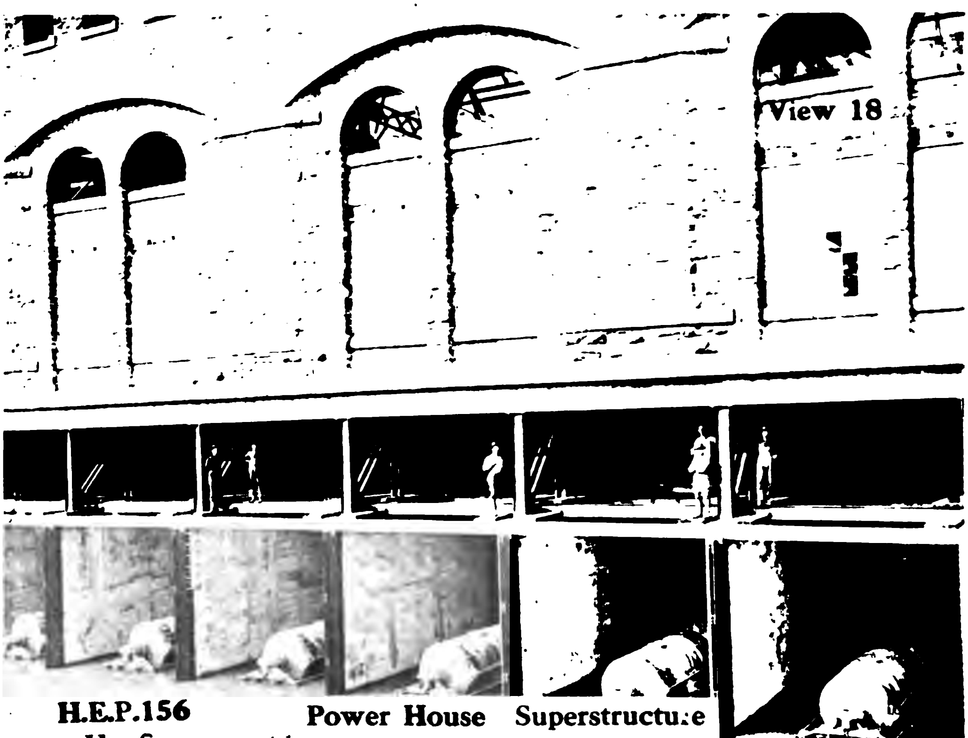
H.v.S.



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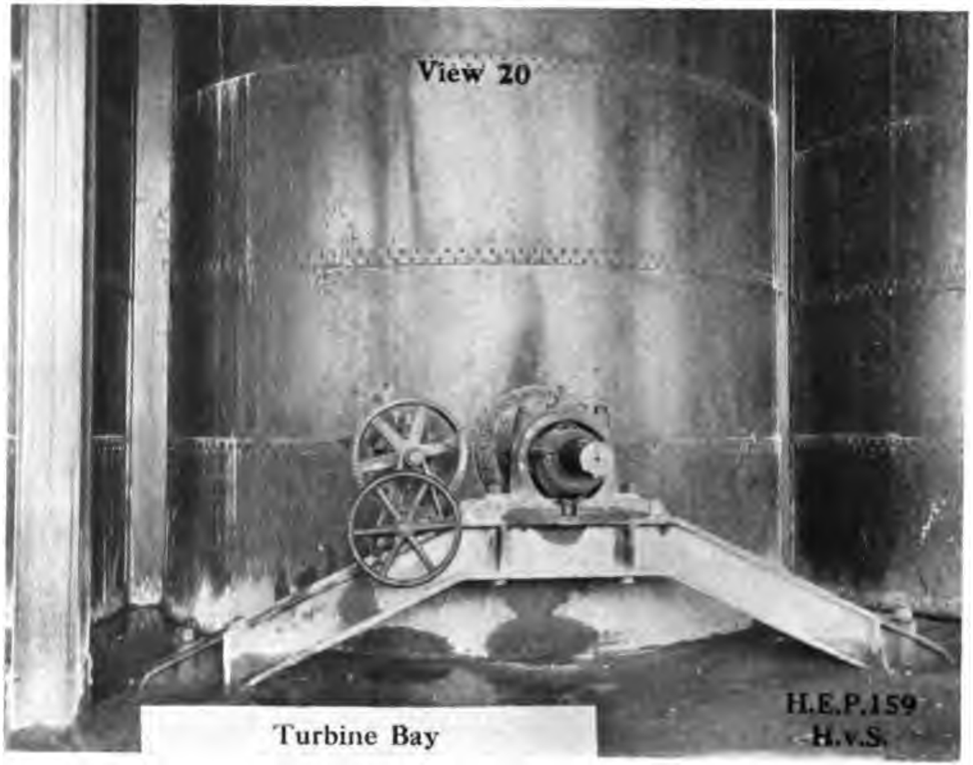
Power House Superstructure
Up stream Elevation

Power House Up stream View

Power House Superstructure
Down stream Elevation



Turbine Bay

ally from 10 to 18 feet wide and from 15 to 40 feet long; their height is controlled by the upper level. The side walls or partitions, Fig. 83, 2, are best constructed of concrete-steel, and must be designed to resist lateral hydrostatic pressure existing when the adjacent bay is unwatered. The downstream end of the turbine bay, Fig. 83, 3, is closed by a masonry or concrete wall or a steel-plate bulkhead of semicircular design with a

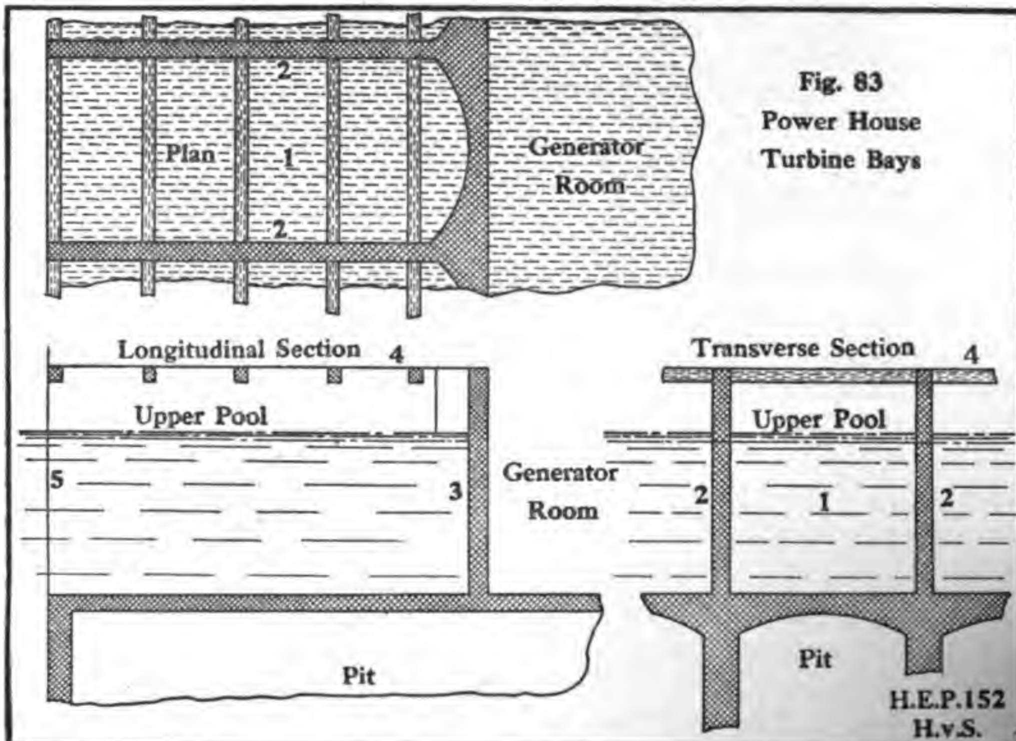
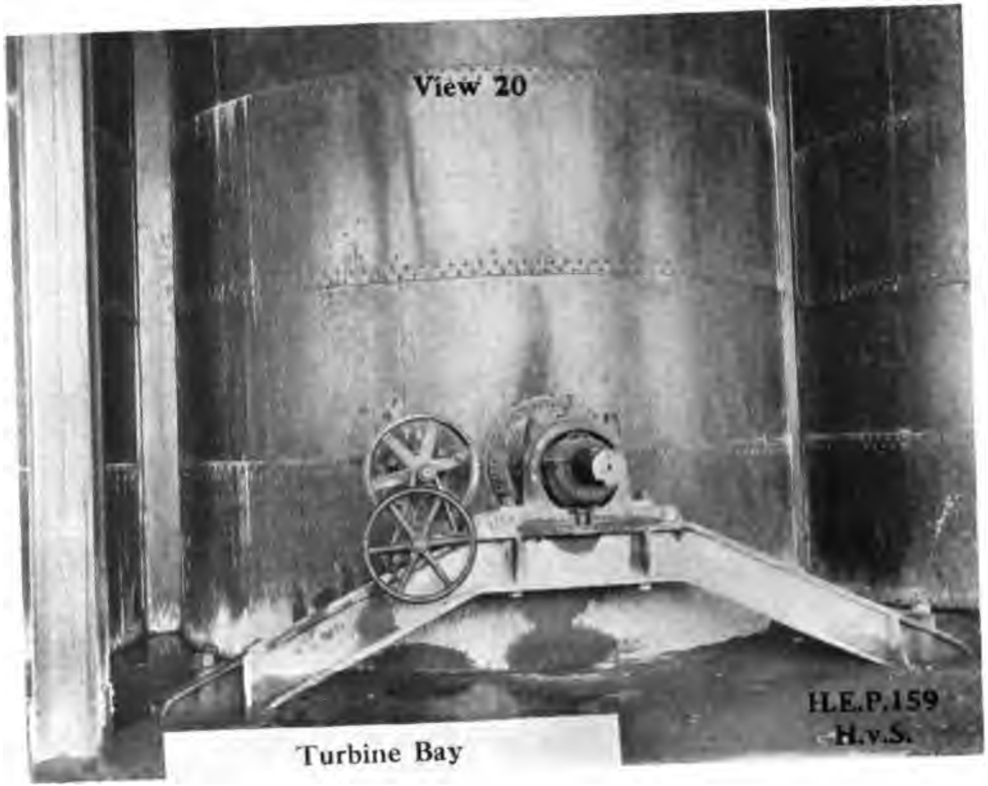


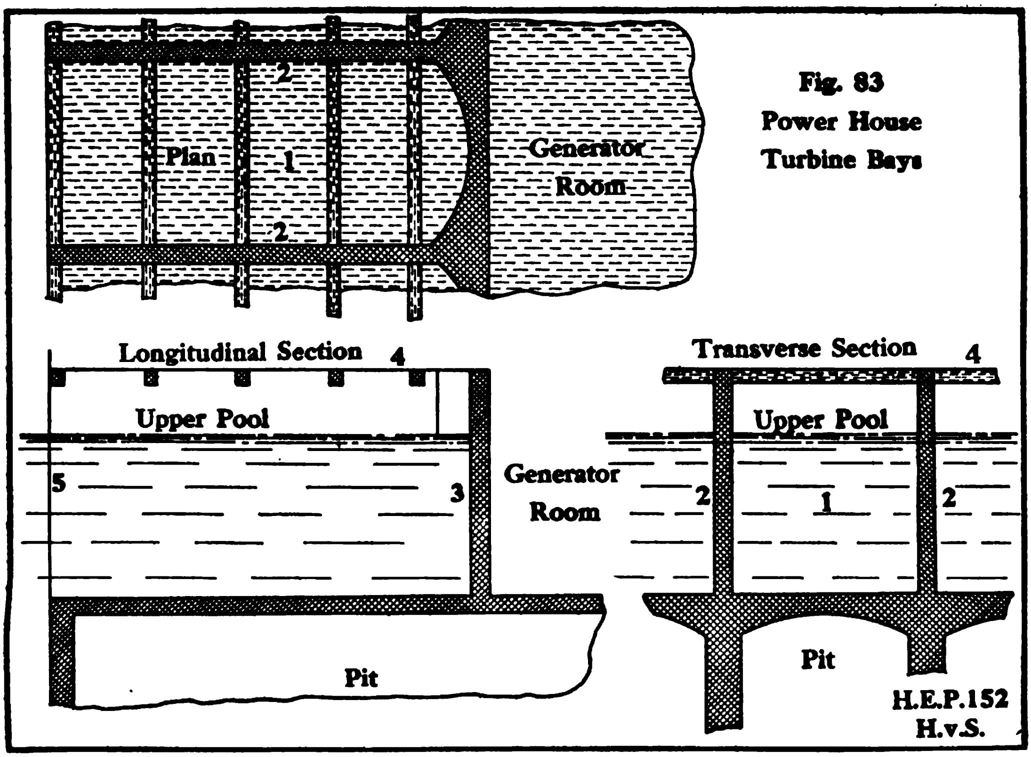
Fig. 83
Power House
Turbine Bays

radius equal to half the width of the bay and secured to the floor and partitions; either of these must resist the hydrostatic pressure due to the upper level head. The partition tops, Fig. 83, 4, are connected by steel members, concrete-steel beams or arches.

Views 18 and 19 are of superstructures of this standard type
upon the substructure shown in Views 16 and 17. The bays are
sioned to accommodate a line of four horizontal turbines; the
feet. In this plant the steel-plate bulkhead was first introduced; a
view of this from the downstream side is given in View 20. All the
from 7 to 20 are of the hydro-electric plant at Sault Ste. Marie,


View 20
H.E.P.159
H.v.S.
Turbine Bay

ally from 10 to 18 feet wide and from 15 to 40 feet long; their height is controlled by the upper level. The side walls or partitions, Fig. 83, 2, are best constructed of concrete-steel, and must be designed to resist lateral hydrostatic pressure existing when the adjacent bay is unwatered. The downstream end of the turbine bay, Fig. 83, 3, is closed by a masonry or concrete wall or a steel-plate bulkhead of semicircular design with a



radius equal to half the width of the bay and secured to the floor and partitions; either of these must resist the hydrostatic pressure due to the upper level head. The partition tops, Fig. 83, 4, are connected by steel members, concrete-steel beams or arches.

Views 18 and 19 are of superstructures of this standard type placed upon the substructure shown in Views 16 and 17. The bays are dimensioned to accommodate a line of four horizontal turbines; the head is 20 feet. In this plant the steel-plate bulkhead was first introduced; a separate view of this from the downstream side is given in View 20. All the views from 7 to 20 are of the hydro-electric plant at Sault Ste. Marie, Mich.,

which was designed by the author and constructed under his supervision (1897 to 1902).

For medium or high-head developments the superstructure of the power house simply becomes a building in which the equipment is placed, consisting of a floor, being the pit roof, and of suitable walls and roof; the turbines are encased, water being supplied to them by pipes or penstocks and discharged into the pit. It is to be deplored that little if any thought is generally bestowed upon the architectural appearance of the power-house superstructure; as a rule, it is of the character of the plainest factory structure; castellated tops or a heavy end tower would be entirely in harmony with the building's purpose and give it a most pleasing appearance; in this respect some of the hydro-electric power houses on the European continent may well be taken as models.

This discussion of power-house types is well illustrated by the following standard designs adapted to different heads and installations. These were all prepared by the author in connection with different projects and some of them have been constructed.

Fig. 84 shows section and elevation of a power house adapted to the lowest head. When less than 12 feet is available, turbines are best placed on vertical shafts, as there is not sufficient head to drown them, the water depth above them should not be less than four feet, and the vertical depth of the turbine runner is about half of its diameter. The foundation is indicated by No. 2 in the figure, the pit by No. 1, turbine bay No. 6, pit walls No. 4, bay partitions No. 7, pit roof No. 6, bulkhead wall No. 8, turbine No. 5, and generator No. 9. The water enters freely into the turbine bay and flows out of the pit. The generator in this case is directly coupled to the vertical turbine shaft, being of what is known as the umbrella type, and is placed on a floor arranged above the turbine bay, supported by the partitions and the bulkhead wall. Note the depression in the pit below the turbine, by which the necessary four-foot water cushion is secured without carrying the entire pit to that depth, which represents some saving in cost. The foundation is the standard pile-bearing type with upstream and downstream aprons. This type of power-house design is available for all low-head situations, the installation being, however, most frequently gear connected, as umbrella-type generators do not as yet form standard electric equipment in this country, though they are being used now more frequently in

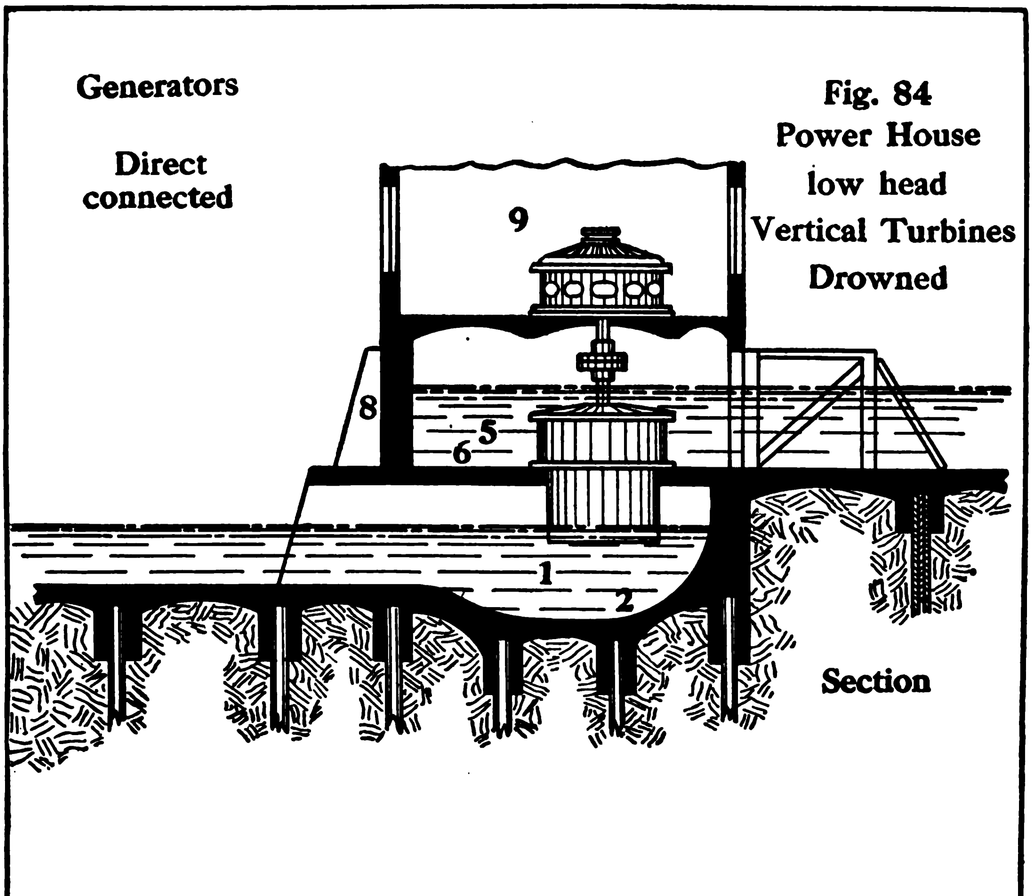


Fig. 84 Power House low head Vertical Turbines Drowned

Europe than the horizontal shaft installation, securing higher efficiency of output. The Niagara Falls plant is so equipped.

Fig. 85 shows the design for a power house with low head in which the turbine bay is arranged, outside of the power building proper, in the forebay. This type is met with frequently in the older plants; it possesses no advantages nor represents economies over the one described in this article; as a matter of fact it adds to the masonry construction. The installation here shown is of a double horizontal turbine drowned, the turbine bay being closed downstream by a masonry bulkhead with a cast-iron head through which the turbine shaft passes; the exit from the pit is in a longitudinal direction to escape interference to free outflow caused by the overfall from spillway.

In Fig. 86 will be recognized the standard low-head design heretofore described, being the arrangement of the plant at Sault Ste. Marie, Mich. The turbine installation consists of two pairs of horizontals; as many as four pairs of wheels have been thus arranged. Note the closing of the downstream end of the turbine bay by the steel-plate bulkhead and compare it with the masonry bulkhead on the design of the previous plant; the cost is about one-third and much valuable space is saved.

Fig. 87 represents a design which is much used in Europe, in which the turbine installation is of the serial type, utilizing fluctuating heads with equal efficiencies. The design differs chiefly from those described in the arrangement of the pit, the discharge being from two levels as the fluctuations of the upper and lower pools may require. The turbines can be only of the vertical type, and abroad the generators are generally direct connected. This design should be used much more frequently than it is in this country, especially on the rivers in the South which are subject to great fluctuations. The foundation is represented by No. 2 of the figure, the pit by No. 1, the turbine bay by No. 5; 3, 6, and 8 are practically bulkheads, 6 being depressed downward to make water seal with lower level at C. The installation consists of a double vertical turbine, 13 and 14, the first discharging upward, the latter downward, and a third vertical wheel, No. 12, also discharging downward; 12 and 13 use the upper pit channel; 14 the lower. The water enters wheels 13 and 14 at 15; 11 is a water thrust bearing; 10 is the generator; 16 is a stop-log gate closing the entrance to the turbine bay. There is nothing complicated about this design nor is it one of greater cost than the

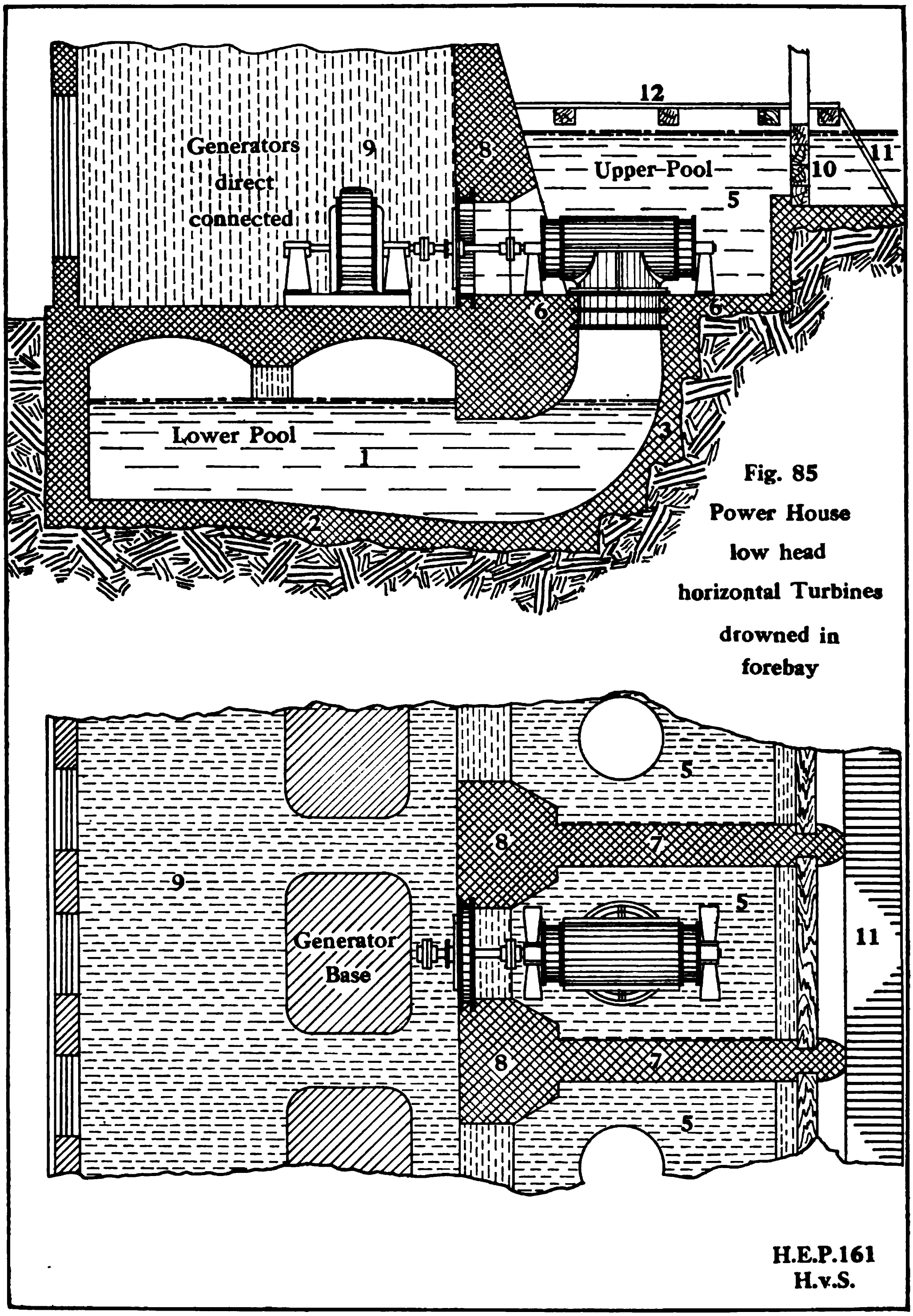


Fig. 85
Power House
low head
horizontal Turbines
drowned in
forebay

H.E.P.161
H.v.S.

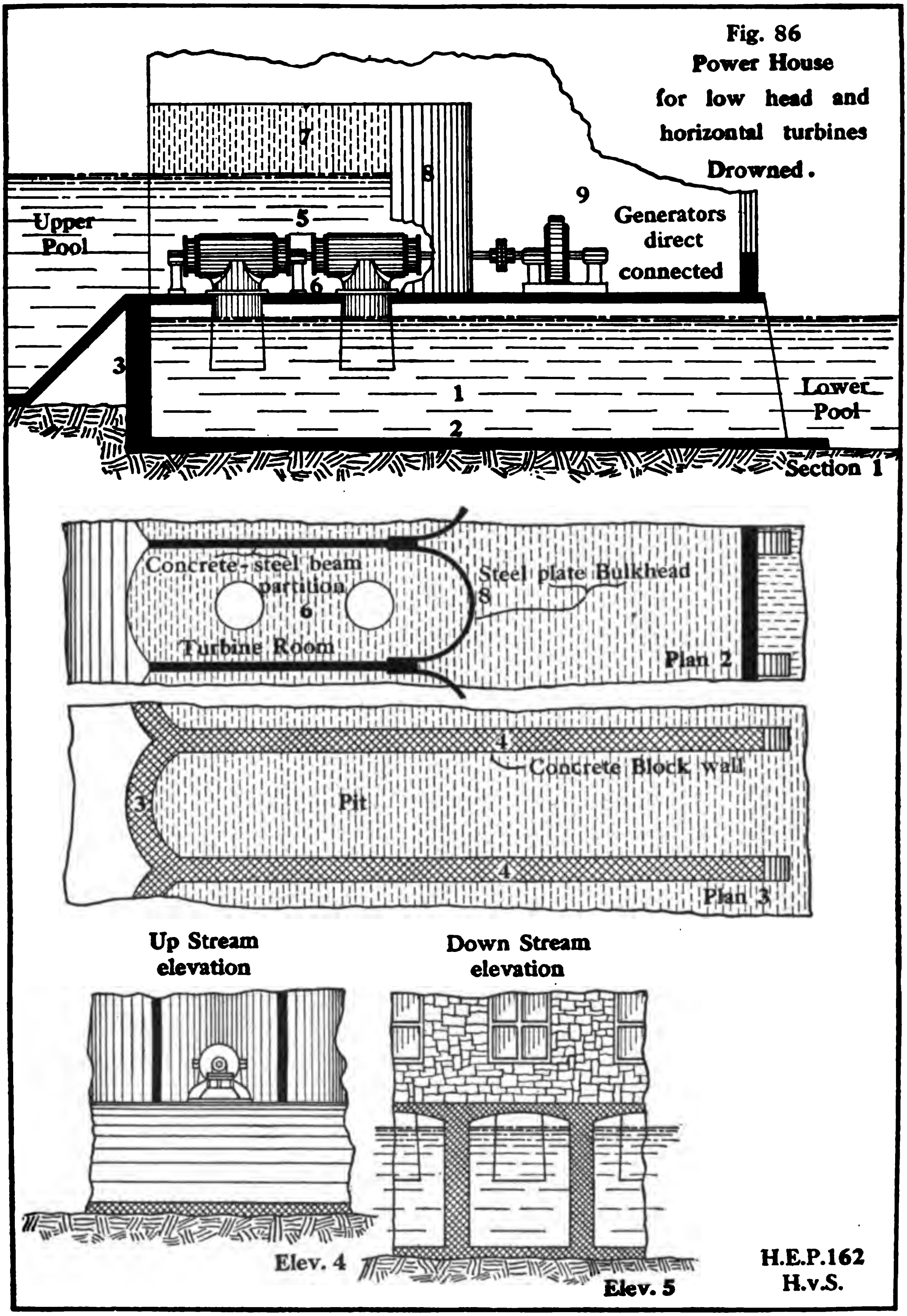


Fig. 86
Power House
for low head and
horizontal turbines
Drowned.
Upper Pool
Generators direct connected
Lower Pool
Section 1
Concrete-steel beam partition
Turbine Room
Steel plate Bulkhead
Plan 2
Concrete Block wall
Pit
Plan 3
Up Stream elevation
Down Stream elevation
Elev. 4
Elev. 5
H.E.P.162
H.v.S.

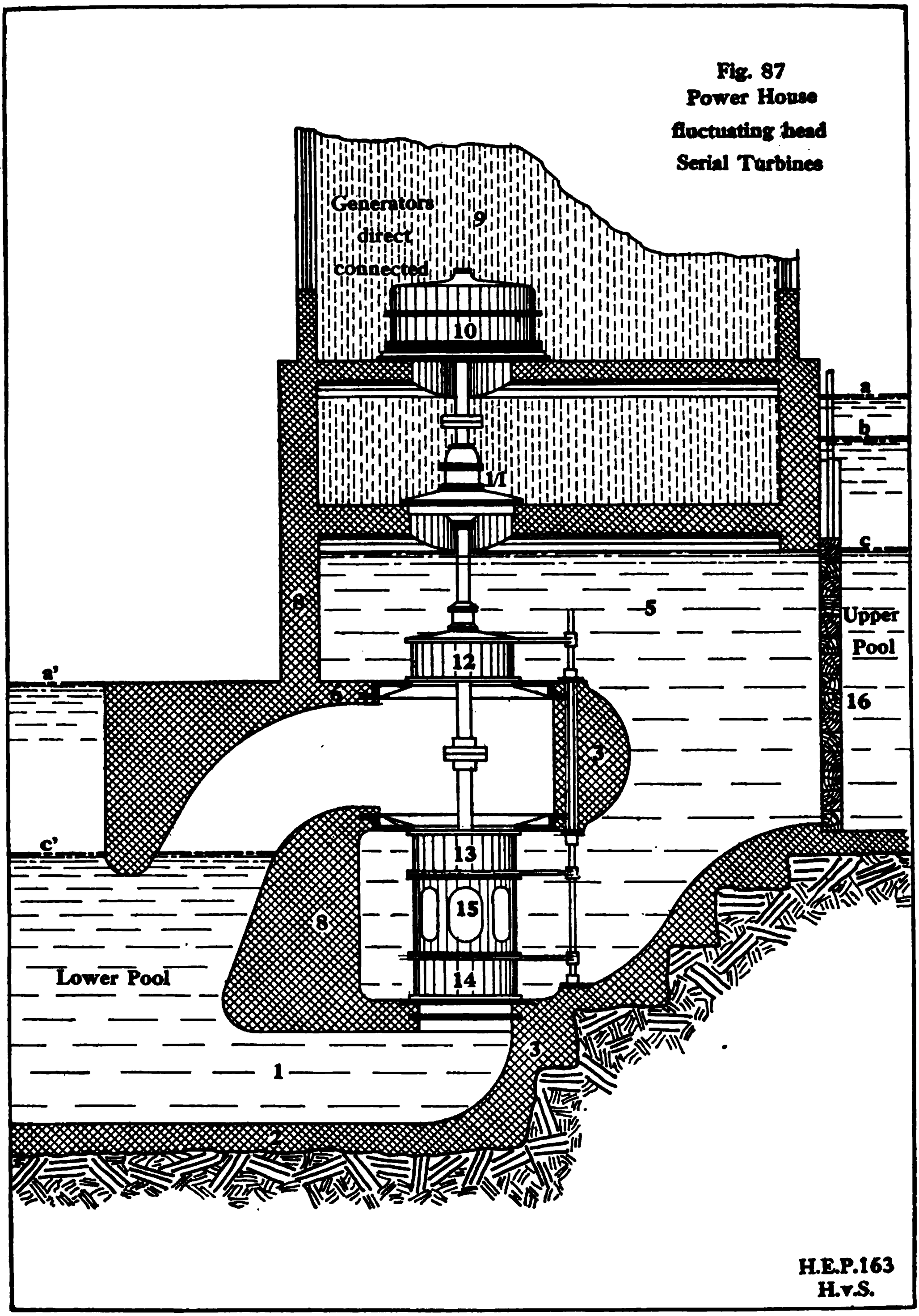


Fig. 87
Power House
fluctuating head
Serial Turbines

others, while it represents the only plan by which an efficient output can be secured from a fluctuating head.

Fig. 88 represents the general practice for low head, the installation being of double verticals discharging by union draft tube. No. 1 is the pit, No. 5 the turbine bay, 13 and 14 the turbines, 11 the thrust bearing, and 15 the generator; 3 and 6 show the forebay wall, 8 the bulkhead.

Fig. 89 represents the general type of power house for medium-head developments; foundation, pit, and operating floor are the constituent parts, and little variety is called for. No. 1 shows the pit, of the same design and construction as in the standard low-head type, unless for high heads, when the accompanying pressures must be properly provided for. The supply pipe enters through the house wall, terminating in the turbine case, from which the water escapes by the draft tube.

This same arrangement answers for the highest heads.

Power-house designing, in the author's judgment, follows too much in old established routes. As the focus and realization of an enterprise comparing, in point of investment and earning value, well with the most important of other lines, it deserves the greatest amount of care. There is a tendency to overdo the heavy dimensioning of walls and allot unnecessarily large areas for the equipment installation; this would be avoided if each part of the structure were independently designed for the duties it is to fulfil. Just as the spillway or a retaining wall is designed, so should the forebay, pit and end walls, turbine bay, partitions, floors, and roof be carefully and individually designed, and the space required for the electric equipment should be laid down from the known dimensions of floor frames and what is required for convenient operating space; anything more than this becomes an expensive superfluity, as it entails more yardage in every wall, more roof and floor areas, and other costly excesses over what is necessary for the purpose.

A desire to incorporate in a power-house design the elements of highest obtainable efficiency with economy of cost of construction and maintenance led the author to develop a power-house design where the interior of a gravity spillway may be utilized as the power station, and by which, it is believed, these ideals can be secured under proper conditions. This design is described in detail and illustrated in Article 78 of this part.

ARTICLE 77. *Appurtenances to the Power House.*—At the entrance of the turbine bays *trash racks* are placed to intercept all floatage. They generally consist of flat steel bars arranged closely to form a rack held in

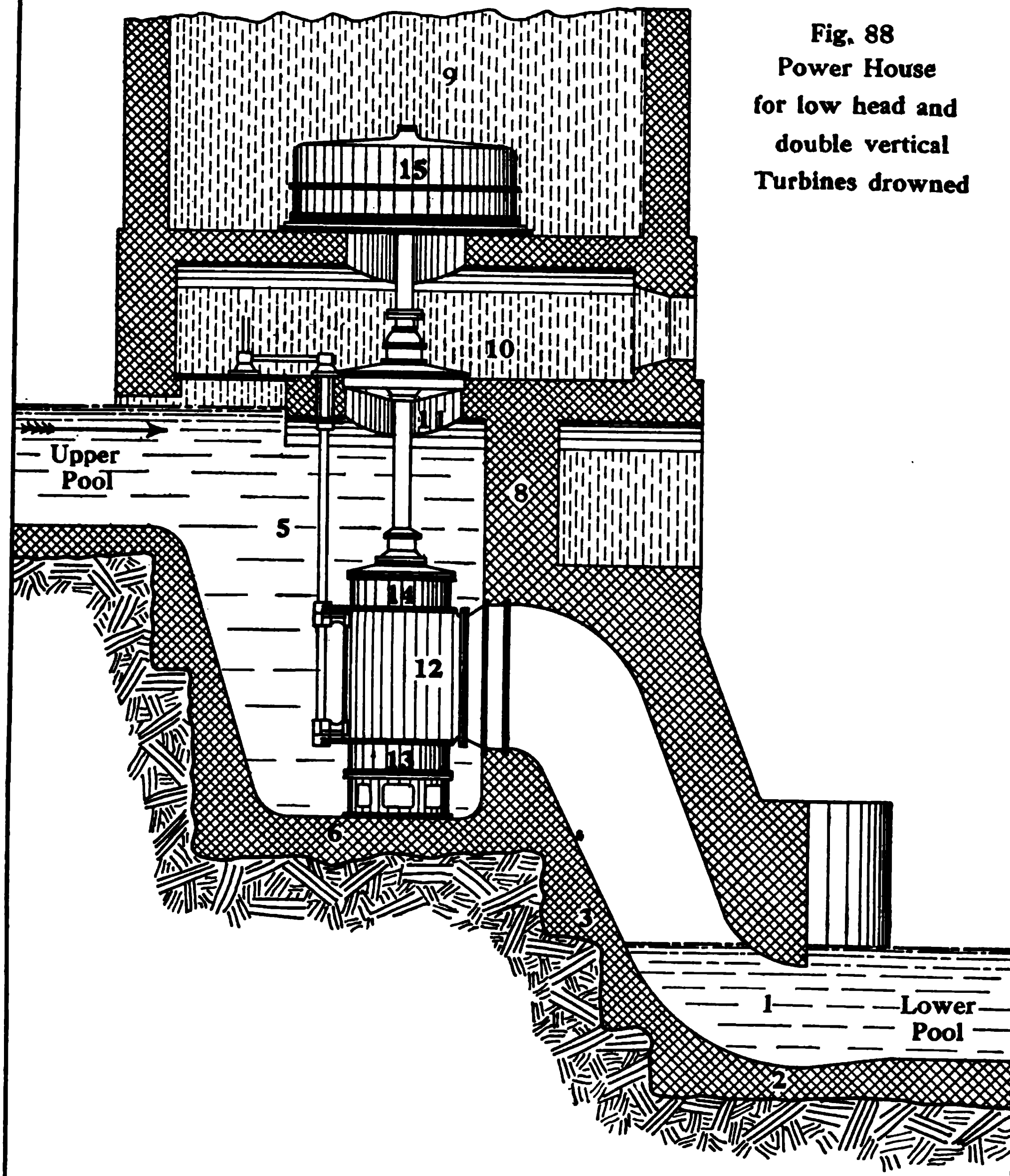


Fig. 88
Power House
for low head and
double vertical
Turbines drowned

H.E.P.164
H.v.S.

a suitable frame; the latter is secured to the structure so that it inclines downstreamward; this general arrangement is shown in Fig. 90; the details depend upon the height of the water and the width of the turbine-bay entrance.

There is not much latitude of design in trash racks. They should be amply strong, and in this regard the winter conditions, as to anchor or float ice accumulations, must be well understood; the former will be treated specifically later on. *An operating platform* is required above the trash rack so that it can be conveniently raked and the floatage disposed of.

The upstream turbine-bay entrance and the downstream pit ends are arranged to be closed temporarily for the purpose of unwatering either chamber and rendering them accessible for repairs to equipment, etc. There are a variety of devices used for this purpose, but, as in the case of open spillway or other parts where gates are required, the author favors the simplest arrangement, such as stop-logs or needles, which are readily handled and quickly renewed; no other device, unless complicated and costly, can be operated more conveniently and cheaply than stop-logs. These have been fully described in Article 70.

A traveller is frequently provided in the power house for the handling of the equipment; it is placed overhead in the operating or generator room and equipped for electric operation.

A machine shop or repair room should always be set aside in the power house. Oil and waste stocks should not be kept in the operating room.

ARTICLE 78. *The Submerged Power-House Design.*—The gravity spillway type is analyzed and a standard design for it detailed in Article 67. From this will be noted that one of the distinct features of this type, as compared with other spillways, is the fact that there remains a spacious interior enclosed by the walls forming the spillway, which, of necessity to serve their purpose, must be safe against rupture and must further be absolutely water-tight, both conditions which render this interior space safe and suitable for utilization of some purpose or other, and why not to place the turbines and the generators therein and operate them?

As has been emphasized on different occasions in this volume, it is highly desirable that the power house be located close to the spillway, so that the conversion of the natural energy into useful work may be carried on in closest proximity to the origin of the natural forces, which

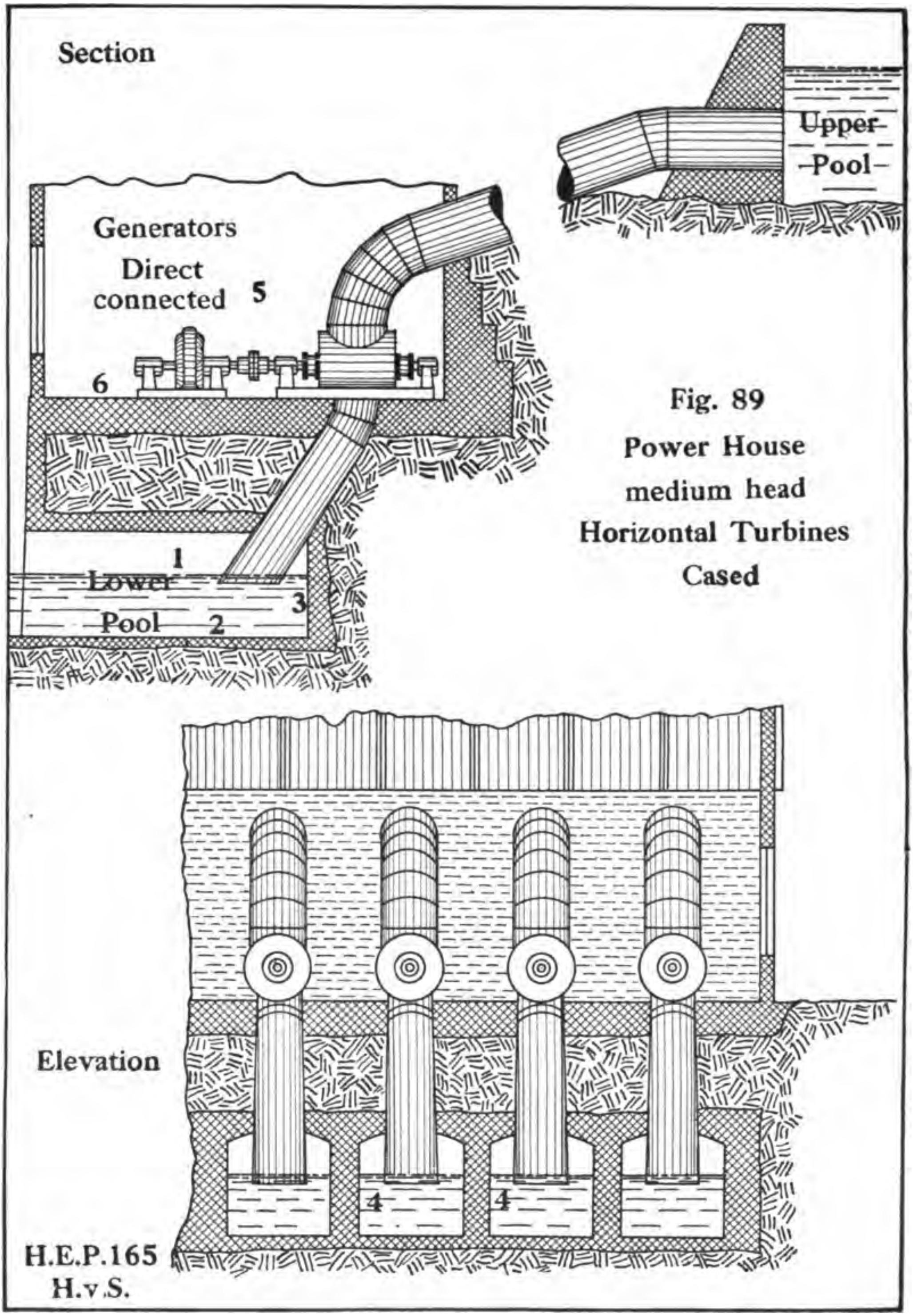


Fig. 89
Power House
medium head
Horizontal Turbines
Cased

is *at the spillway*, unless in the case of the necessity of a diversion programme. It is the aim of all industrial enterprises to locate as near as practicable to the point of raw material; that is exactly the case here, as it costs as much (and generally more) to transport water and head as any other class of staples; not only is it a costly undertaking, but, much more to the point, it is a wasteful process, this transporting water

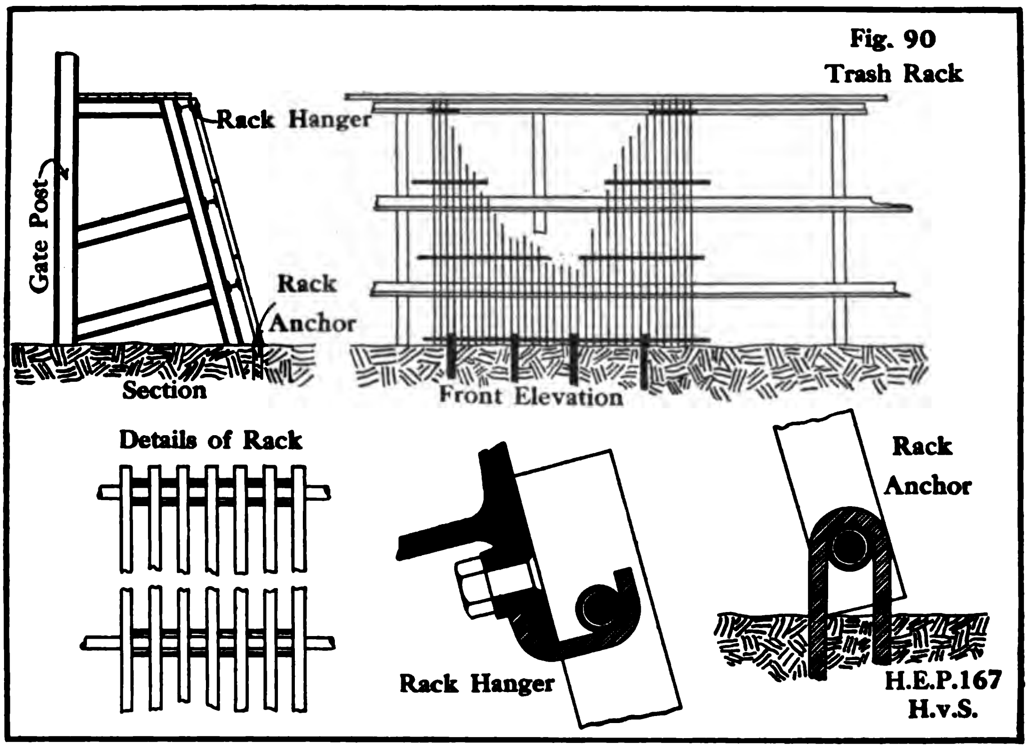


Fig. 90
Trash Rack

for any distance from the spillway to the power house, as water does not move unless fall or head is expended and that is lost forever. The spillway has a secure foundation, and it, with the walls, practically forms a house, excepting that it is not of the conventional shape; its walls incline toward each other and jointly form the roof; at any rate, it represents a substantial structure, and if it is sufficiently roomy there is no reason why it could not be utilized for the purpose of a power station. The question of dimensions is readily determined when the fall and power output are known. Generally speaking, the submerged power house can be arranged when the fall exceeds 25 feet and the fluc-

tuations of river stage remain within five feet; the output does not generally put a limitation on the use of this design, as the entire spillway length is available.

Fig. 91 shows the general arrangement, which can be varied to suit different conditions.

In the section No. 1 shows the walls of the spillway, the deck and apron and crown; one of the partitions arranged transversely of the spillway structure, as appears more plainly in the partial elevation and longitudinal section; these partitions, or they may take the form of steel frames, need not occupy the entire transverse space from deck to apron, their central portions can be omitted, leaving an arch opening, which, if need be, reinforced by a steel member, represents the strength of support for deck and apron in the same degree as if it were solid. This statement needs no more argument than the fact that a wall containing a door, properly framed, will support the same weight safely as it would if it were solid.

No. 2 shows a vaulted chamber arranged longitudinally in the interior of the spillway, its floor of concrete-steel arches resting upon transverse walls, really the bottom sections of the partition widened in section, and its walls and ceiling being formed by one continuous dome-like structure, generally of hollow brick tile, or of ferro-inclave. A considerable space is left between this chamber and the shell of the spillway structure, allowing of free circulation of air currents around it. The length of this interior chamber depends upon the number of power units, generally one unit can be installed between two spillway partitions. The power chamber or operating room, or by whatever name it becomes known, is continuous and need not be subdivided for purposes of design of construction, though it may be desirable to partition off a repair shop and perhaps an office room. This vaulted room may be compared with the superstructure of the general power-house design, while its supports are identical with the ordinary substructure containing the pits and turbine bays. All of this is readily recognized from the sections shown, and, as a matter of fact, the structural arrangements are subject to such a variety of designing as may be suggested by different conditions and requirements.

No. 5 shows the feed-pipe intake passing through the spillway deck; it may be as shown, or higher up; the feed pipe may lead in straight or with a quarter turn,—all matters of detail for particular cases; at any

rate, the water is drawn from the upstream side of the spillway, led into the turbine case, which, as here shown, is that of a vertical shaft wheel; it may be of the horizontal type, in fact of any of the installations which will appear further on as part of the discussion of hydraulic equipment, all depending upon the required output and available space. In a plant recently designed the installation consisted of a line of two 54-inch turbines on one horizontal shaft driving one generator.

No. 6 is the supply pipe, and at

No. 7 is shown a valve gate by which the flow in the pipe is fully controlled; this is also a subject of detail, offering no difficulty of solution.

No. 8 is the turbine case, already discussed.

No. 9 shows the draft tube passing into the pit. Note the depression in the pit excavation below the draft tube, also the depression of the outflowing water surface at No. 4; this will actually exist whenever water is passing over the spillway, excepting during storm flow, representing the siphon action of the water as it passes the downstream opening of the pit, where its energy is that due to its mass and the fall. If the overfall on spillway crest is six inches deep, the volume per linear foot of spillway is approximately 1.2 cubic foot per second, which with a fall of 42 feet, that of the section shown, represents a theoretical energy at the toe of the spillway of about 3000 ft. pds., where it meets the tail water passing out, and, instead of exerting its force on the river bed material or the spillway apron, it is turned into the harmless but extremely useful work of accelerating the motion of the tail water, thereby lowering it below the draft tube and incidentally increasing the effective working head on the turbine. The conditions which give rise to this phenomenon would usually combine to cut down the effective head of the plant by raising the lower level in the tail-race or pit. This is not only based upon theory, which is sound enough, but has been actually observed, and the author hopes to find opportunities, before very long, to make tests and measurements of the actual work of the overfalling water expended on the escaping tail water.

No. 15 shows two waste flumes arranged close to the supply-pipe intakes, by which the accumulation of sediment near these intakes can be avoided. As already stated, there is no objection to the arrangement of the supply-pipe intake at a higher level. The intake entrance is protected by a trash rack operated from a platform on the spillway crest. Raking may also be arranged from the interior of the spillway.

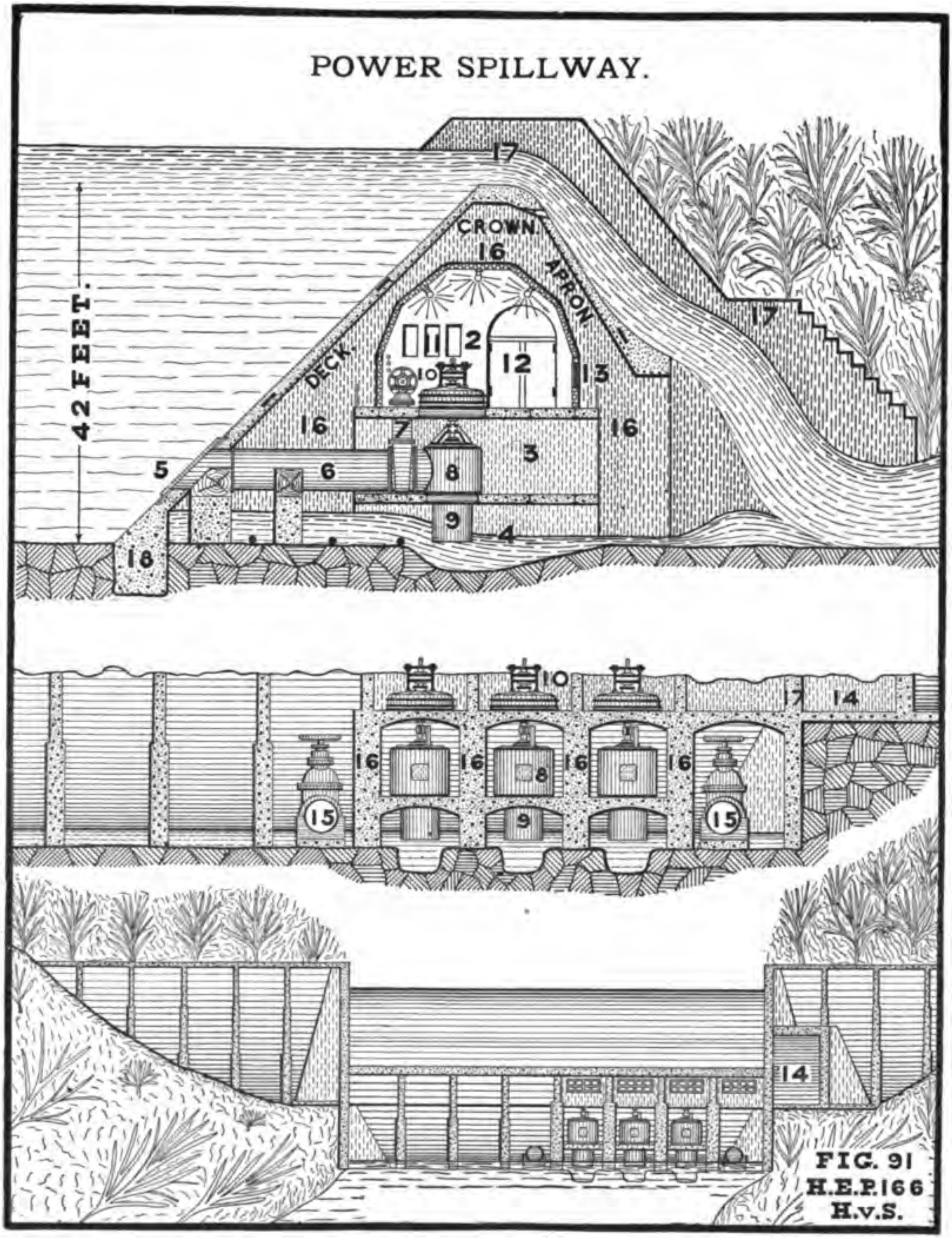


FIG. 91
H.E.P.166
H.v.S.

No. 14 shows the entrance to the submerged station, which is from the downstream side of the spillway through the abutment, entirely safe and as convenient as any transportation arrangement can make it; a railroad track may be brought to this entrance or carried into the station, a trainload of equipment may be taken into the station,—simply a question of available space.

No. 12 shows the end door and entrance into the operating room.

No. 13, the windows, in this case of ordinary rectangular dimensions, because the apron is of the partial type; were it full, the light would be supplied through ship lights; there need be no scarcity of light, daylight and artificial; when water overflows the spillway the refracted light, through the water, is plentiful and most beautiful.

Abundance of circulating air is secured in this chamber; if anything, too much of it when overflow sucks air through the vents under the crown from the interior of the spillway, thus creating a considerable air current.

The equipment as herein shown in No. 10 consists of a vertical or umbrella generator, an exceedingly economical machine both as regards material and dimensions, and of high efficiency; witness the equipment of the Niagara Falls plant and many of the most recent and complete of the European installations. The generator may be of the ordinary type coupled to the shaft of a horizontal turbine, or a pair; the vertical is somewhat more economical in space. There is plenty of room for the hydraulic governor, the exciter and switchboards, and the cable galleries which lead out to the abutments and thence to the transformer house or transmission line.

No. 3 shows the space available for the handling of the turbines in placing or removing them; they are passed through the floor and the draft tubes similarly through wells arranged in the lower floor.

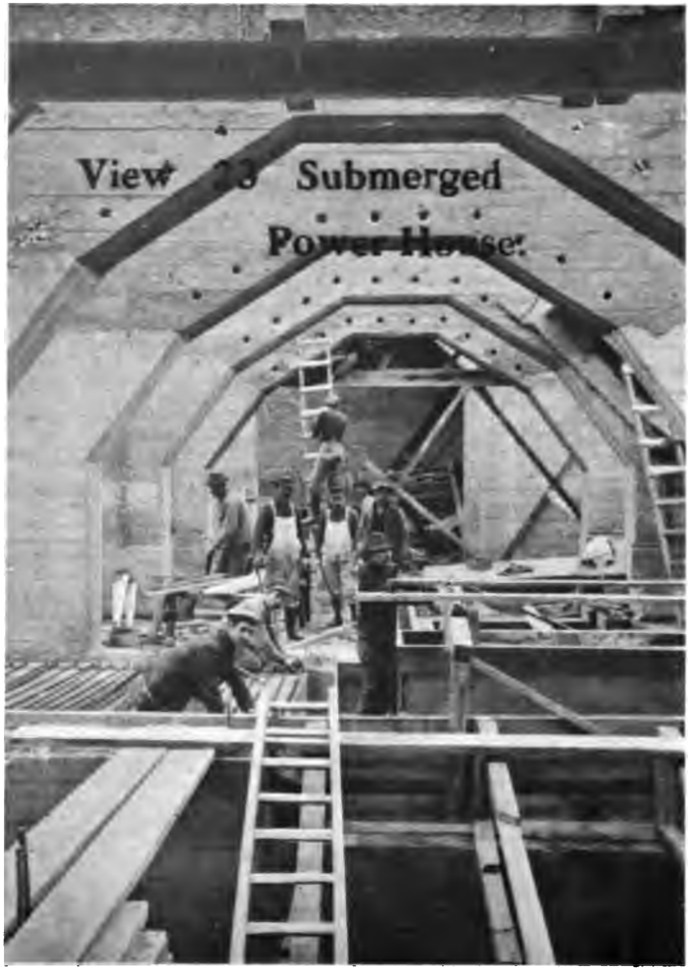
No. 17 is the abutment which is designed to protect the entrance leading into the station on the other side of it against any of the overfalling water.

The advantages represented by the submerged power house may be enumerated as follows:

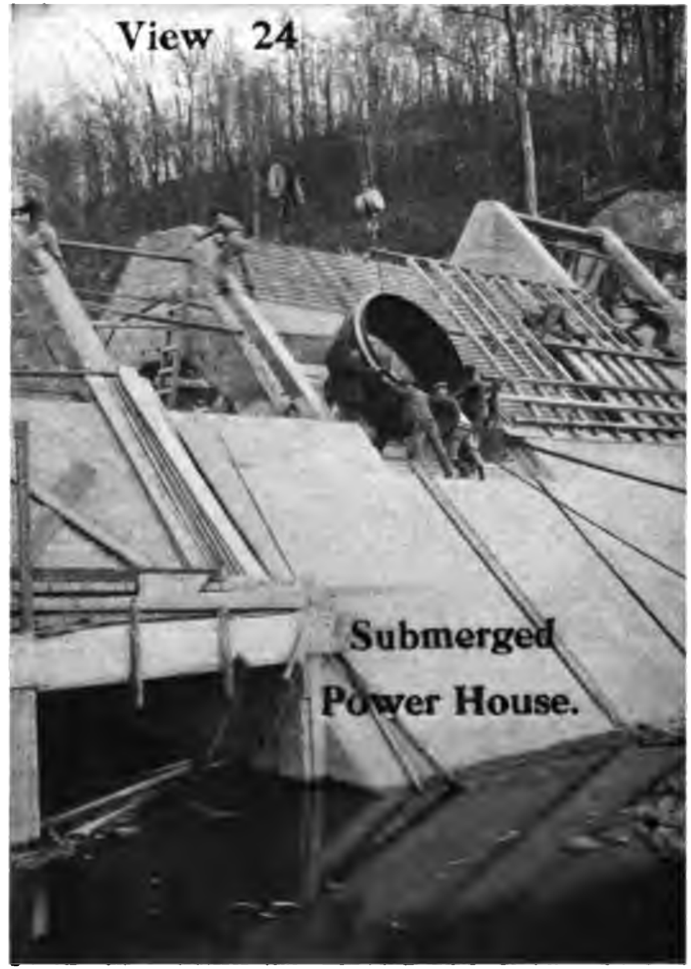
First, it secures the highest obtainable hydraulic efficiency of the available flow and fall, certainly exceeding that of any other arrangement, unless it be in a power house at the end of the spillway.

Second, it represents the greatest obtainable economy in power-house construction, and in this respect admits of no rival and no exceptions.

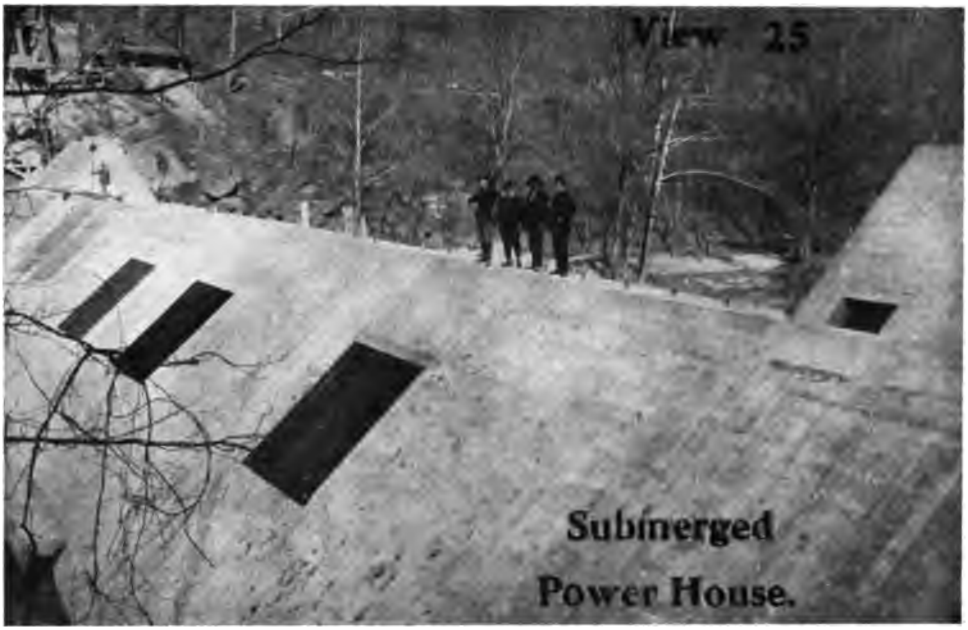
View 21



View 23 Submerged Power House.



View 24 Submerged Power House.

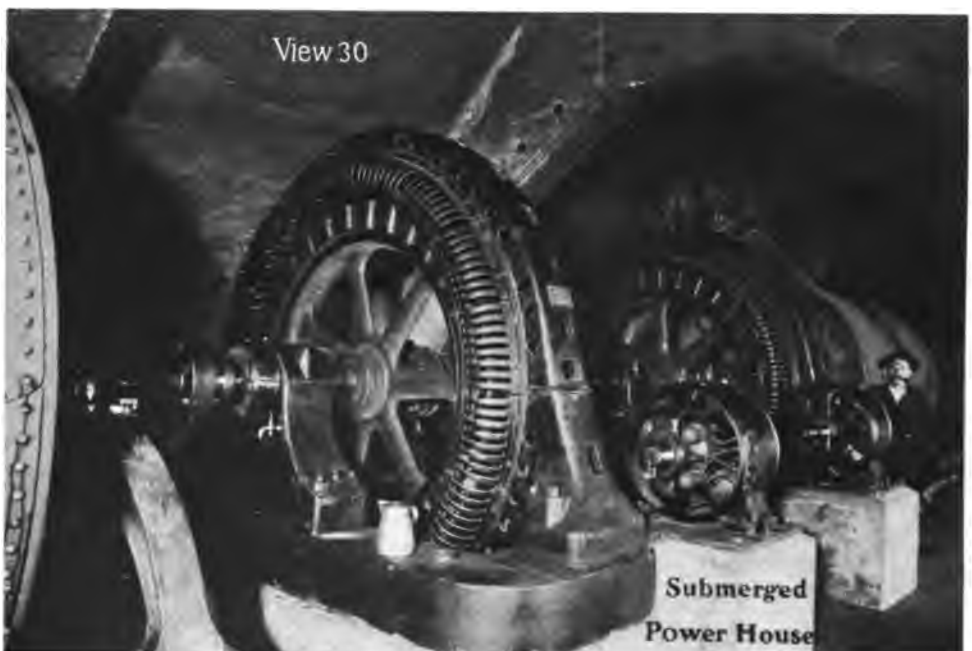

View 25
Submerged
Power House.


View 26
Submerged
Power House.


View 27


View 28
Submerged


View 29
Submerged
Power House.


View 30
Submerged
Power House

Submerged Power House

View 32

Even the power house at the end of the spillway requires a separate structure, unless the spillway proper is shortened for that purpose, when it represents a far more expensive arrangement than a separate power house, because the storm overflow will be much higher and flood more land, etc. The saving represented by the submerged power house should be considerable in any case as compared with the separate power house, but, even if the cost were the same, the advantages secured under the first item represent a substantial value.

Third, the submerged power house is absolutely safe from lightning danger; it cannot be struck, nor damaged by fire in any manner.

Fourth, the power installation may be of gradual growth without the necessity of erecting an unnecessarily large building at the first. This is frequently a very pertinent point, that, while the spillway must be completely built, the power house might be economized on until the market calls for the output; but the separate power house must, as a rule, be as completely constructed at the beginning as the spillway and dam, even though at first only a part of the power installation is to be placed. In the submerged power-house type, the power house is practically constructed when the spillway is in commission, the interior detail arrangement can be made just as conveniently and economically at any time thereafter, perhaps with considerable saving when electric power is later on available from one or more units.

Fifth, the submerged power house can be constructed—or, more appropriately speaking, arranged—irrespective of the condition of the river or the season of the year, which very frequently is an important matter when the completion of the plant has to go over a season because of high water or serious cold. It will be simply interior finishing.

At the time this publication goes to press, the first of this type of submerged power stations has been completed and is in operation. It is located on the Patapsco River some 15 miles from Baltimore, Md., and is easily reached from that city by the B. & O. R. R. going to Illchester, Md., or by the electric Interurban to Ellicott City, Md., the plant being only a short distance from either point.

The development is the project of the Patapsco Electric and Manufacturing Company, of which Mr. Victor G. Bloede, of Baltimore, is the President and Mr. Otto Wunder, of Ellicott City, Md., the Superintendent. The Patapsco River is one of the oldest water-power streams in this country, and one of the earliest developments at Ellicott City, consisting

of a spillway and diversion canal and constructed in the last century, is still in commission operating some large flour mills located at Ellicott City, and it seems peculiarly fitting that the latest development of hydro-electric plant types should find its cradle on this stream which is so full of water-power history.

The Patapsco Electric and Manufacturing Company now operate two hydro-electric plants on the Patapsco River, the one here referred to with the submerged power station, and an older plant some two miles above this on the same stream. Mr. Otto Wunder has charge of both of these plants, and is therefore well qualified from the view-point of the practical station operator, the man who finds or misses the faults and merits more readily than the designer and constructor, to draw comparisons between the usual type of the separate power house with this submerged station located in the interior of the spillway, and Mr. Wunder is a man whose talk is plain and convincing because he knows what he is talking about.

Views 21 to 32 are of the Patapsco plant.

View 21 shows the construction of the spillway, which was executed by the Ambursen Hydraulic Construction Company, of Boston, Mass., who make a specialty of this type of spillways and to whom the author is indebted for these construction views. In No. 21 the spillway partitions are being formed and the openings for the interior chamber are plainly visible.

No. 22 is an end view, showing the rock bank against which the abutment is placed and through which entrance is gained into the interior station.

No. 23 is a longitudinal view of the interior, giving a good idea of its dimensions; the floor of the station room is being placed.

No. 24 shows the placing of one of the turbine supply-pipe collars in the upstream face, the deck, of the spillway; it also reveals the waste flume, an underflow sluice, in the lower left-hand corner, by which part of the stream's flow is controlled during construction.

No. 25 is an upstream view of the completed spillway with the supply-pipe openings, which are covered with trash racks and gates.

No. 26 is a downstream view, showing the partial overfall apron and the openings in which the windows of the station are placed.

No. 27 contains a downstream view of the completed structure, showing the water passing through the underflow sluice; the windows

of the power station are plainly visible here; they are of ample size, making the interior station as bright as day.

No. 28 is of the interior, with two of the power units installed and operating; the double horizontal reaction turbines are in cases; the supply pipe enters the case at the top.

No. 29 is a similar view taken from the other end, and

No. 30 shows the two revolving field alternators and the exciters.

Views 31 and 32 are of the spillway with water flowing over its crest, this plant having already passed through two of the highest of the known flood rises on that stream. Under this condition the overfall is carried safely down the apron, the light shines through it, and the appearance of this spectacle from the interior chamber is not readily forgotten. The station is absolutely safe from leakage or any moisture whatever.

At the present time, November, 1907, two other plants of the submerged power station are being constructed, one at Delta, Pa., and the other on the Big Horn in Wyoming; these are both 60 feet high and will develop about 1500 kilowatts output. Arrangements are also about made for the construction in Canada of a similar plant, in which the dam will be 800 feet long and 45 feet high, the submerged station to contain probably twenty turbine units, to which will be coupled pulp grinders. This spillway will practically contain a mechanical pulp-mill, all the operations of preparing the pulp wood, grinding the same by direct connected pulp grinders, running the pulp through wet machines, and manufacturing it into pulp sheets ready for shipment, will be carried on in the interior of this spillway; and the cars will be switched and loaded in the submerged station, while it is likewise seriously intended to provide living quarters for the operatives in it, as the location is remote from any present settlement. All this is entirely feasible, and the realization of such a plan represents a very large saving as compared with the erection of a separate pulp-mill outside of the spillway, diverting the flow to a separate power house, and erecting lodging-houses, etc.

Pumping plants for water supply can be conveniently thus placed in the interior of the dam which creates the reservoir, and electro-chemical plants could be likewise thus arranged with a considerable saving in cost.

CHAPTER IX

EQUIPMENT

WHILE it is the purpose of the structures described in the previous chapter to collect and make available the natural forces for their ultimate utilization, the realization of it all is left to the agents by which the energy of falling water is made to do the work and light the way of man many miles away. The virgin power is hydraulic, which by hydraulic turbines is converted into *mechanical* energy, the latter by electric generators is changed into *electric current*, which is finally *transmitted* to the market; the equipment, therefore, is that representing these three stages of conversion, and will be treated in this chapter as that required for hydraulic, mechanical, electric, and transmission duties.

The presentation of this subject of equipment will be found analogous to that of the former topics,—that is, with and for the purpose of presenting the practical features, only so much of the fundamental theories being developed as seems essential to a clear understanding of the results to be sought by the employment of this equipment. Designing and construction of equipment are necessarily confined to the general outlines of modern types, and the same is true of the output. As a proper treatment of the different makes of equipment, even if confined to American usage, is beyond the desired scope of this work, reference to any specific type has been avoided; when this rule is apparently set aside, it is in the case of a special and patented device.

ARTICLE 79. *Hydraulic Equipment, Theory.* — Water-power is rendered serviceable as mechanical energy through the agency of hydraulic turbines. The energies of water are of position, *potential*, and of motion, *kinetic;* the first is expressed by gravity through the weight of the mass acting under pressure due to fall; the second by impact due to the velocity of flow; the sum of the two forms *hydro-dynamic energy.*

Dynamic energy is produced whenever the direction or the velocity of a flowing stream or jet is altered; it may find expression as *impulse,* which is pressure forward in the direction of the flow, or as *reaction,* the pressure backward in a direction opposite to that of the flow, both forces

being those existing in all bodies under stress, action and reaction, which when not restrained are equal in intensity.

The potential energy is expressed by the product of the weight of the mass and the height of its fall; this may be utilized to do work; one cubic foot of water falling ten feet represents 625 foot pounds, and in this form water-power energy was employed during the earlier periods of our civilization by letting water fall into the buckets of the overshot and breast or middleshot wheels, which rotated around a horizontal axis, from which the resultant power was taken by gear connections to mechanical drives; about fifty per cent. of the initial energy was thus realized for useful work.

The kinetic energy is that of impact due to the velocity of flow; it may be converted into useful work by the aid of undershot, paddle, and current water-wheels, in which the flowing water strikes the vanes secured to the hub of a wheel; not to exceed 45 per cent. of the original energy may be realized.

The sum of potential and kinetic energies represents the hydro-dynamic equation; the first is the pressure, the second the velocity; combined they represent the material energy created by water-power. While a stream passes as a continuous volume through confined channels, its dynamic energy remains intact; if the velocity increases, the pressure is diminished, and vice versa, the aggregate remains the same and available to perform work; if its free flow is altered in direction, without shock, the total energy passes into the new direction; but shock is overcome by impact, which is work done, entailing the expenditure of some of the available energy. If the obstruction to its free passage is of a movable device, energy, expressed as impulse or reaction or both, may, in overcoming the obstacle to its free flow, be transferred to the movable device, which takes up this energy, or so much of it as is not required to overcome mechanical resistances. This is the principle of the turbine; the direction of a stream is altered by interposing movable vanes, and the force of the stream which would resist such a change REACTS upon the vanes and through their motion is converted into mechanical work.

Dynamic energy finds its final expression in weight; the absolute unit is $1 \div 32.16$, being the force which will move one pound one foot in one second. Energy of water is its *mass*, the product of its weight and the absolute force unit. When water moves inertia is overcome and motion is created and maintained by the acceleration of gravity in

response to a certain fall and flow, expressed in feet per second. The *momentum* of the moving water is the product of its mass and the velocity with which it flows.

F represents the theoretical force of the momentum of flowing water;
W represents its mass;
w represents the weight of one cubic foot of water (= 62.5 lbs.);
g represents acceleration of gravity (= 32.16);
V represents velocity of flow in feet per second;
h represents fall in feet; and
A represents the cross-sectional area of the flowing stream or jet in cubic feet;

$$F = \text{mass} \times \text{velocity}$$

$$= w\,A\,V \div g \times V \qquad = w\,A\,V^2 \div g.$$

Static pressure, P = w A h, and with V expressed in h, dynamic force F = 2 w A h or double that of static pressure.

In the following discussion dynamic force will be expressed by

$$F = \frac{W}{g} V.$$

A stream enters a plane, Fig. 92, A C, at A, without shock or change of flow section; its entry is in the direction of A B, but it is forced continuously away from this into a new direction, A C. At the point of entry the stream has no tendency to deflect, and it therefore requires some retarding force to cause this deflection, which in turn sets up an accelerating force in the stream acting continually in a direction at right angles to its original motion. At C, or at any other point along the plane A C, the deflection C B = A C sin e, and the force which has caused it is represented by the product of C B, as the measure of velocity divided by the period of time, or AC sin e ÷ t, and the pressure of the mass of water passing over AC per second.

If the plane AC, Fig. 93, is not fixed, theoretically the accelerating force AC sin e will act upon the plane in direction CB, and AC will occupy the position DB when the stream arrives at C, and if another stream then enters at D, and so on, the plane will continue to move in the direction BE. Impact wheels representing the earliest turbine types were

designed on this principle, from 16 to 20 rectangular blades or paddles being fastened to a wheel at inclinations of 50° to 60°, the water striking the paddles at about a right angle.

AC, Fig. 94, is a curved vane; the stream enters at A without shock and leaves it at C; the force F in the stream's original direction equals the impulse less the reaction, or

$$
\begin{aligned}
F &= I - R' \text{ (the component of R)}, \\
R' &= R \cos x, \text{ and, as} \\
I &= R = \frac{W}{g} V, \\
F &= \frac{W}{g} V \,(1 - \cos x).
\end{aligned}
$$

In Fig. 95 the same conditions prevail as in the previous example, excepting that the angle at the exit exceeds 90° and

$$
\begin{aligned}
F &= I + R', \\
R' &= R \cos 180 - x, \\
&= R \cos x,
\end{aligned}
$$

and, as in the previous case,

$$F = \frac{W}{g} V \,(1 - \cos x).$$

This principle was utilized in the impact wheels with curved vanes, also called tub wheels, or the French roues en curves, and the German Kuferraeder, of which class the Burdin turbine realized probably the highest efficiency.

In Fig. 96 the vane is bent completely back, so that the direction of the exit flow parallels that of the entry and

$$F = I + R = 2 \frac{W}{g} V,$$

or double that of the normal value of F. It must be noted that F represents the force in the direction of the flow of entry and that the vanes considered have been fixed.

In Fig. 97 the vane has its individual motion U, and the velocity of the stream along the surface of the vane and relative to it becomes V − U, while the force of the stream in the direction of its flow is as before excepting as to the value of the velocity, or

$$F = \frac{W}{g} (1 - \cos x)\ (V - U).$$

Fig. 98 is an analysis of the theory of the entry and exit of the water in contact with a moving surface.

V is the relative entry velocity;
e is the entry angle between tangent to point of entry and normal to direction of the vane motion;
V′ is the absolute velocity of entry, and
e′ the angle of its direction with that of the vane's motion normal;
U is the velocity of the vane;
V — U, effective velocity of the water along the surface of the vane;
V, the relative exit velocity;
X, the exit angle of tangent to point of exit and normal to motion direction of the vane;
V″, the absolute exit velocity.

Fig. 99 completes the consideration of the characteristics of flow along moving vanes (not rotary), exemplifying the determination of the total force acting in the direction of the motion of the vane, from the components of impulse and reaction.

D marks the direction of the flow and
U marks the direction of the vane's motion;
e is the angle formed by these two;

the available force is $F = \frac{W}{g} V$,

the impulse component $= \frac{W}{g} (V \cos e)$,

the reaction component $= \frac{W}{g} (V \cos x')$,

and $F = \frac{W}{g} (V \cos e - V_, \cos x')$.

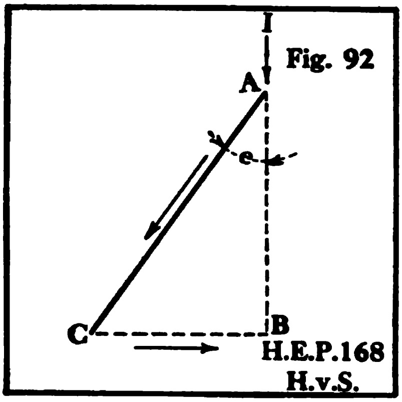


Fig. 92

H.E.P.168
H.v.S.

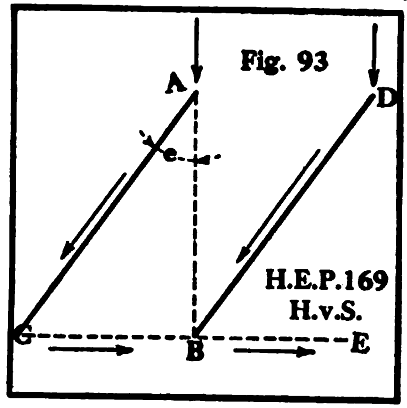


Fig. 93

H.E.P.169
H.v.S.

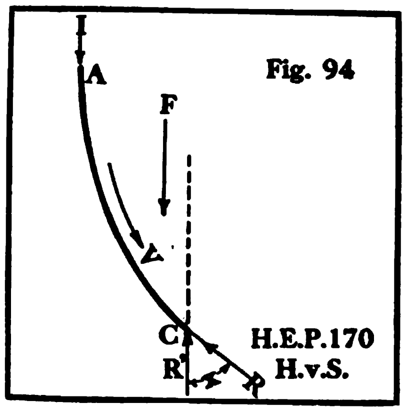


Fig. 94

H.E.P.170
H.v.S.

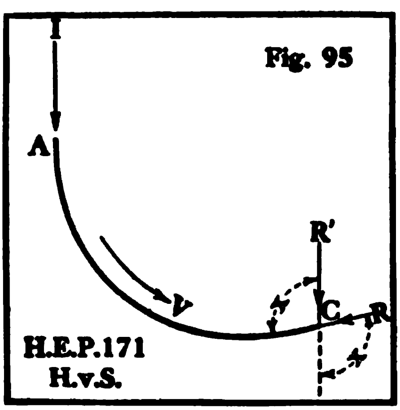


Fig. 95

H.E.P.171
H.v.S.

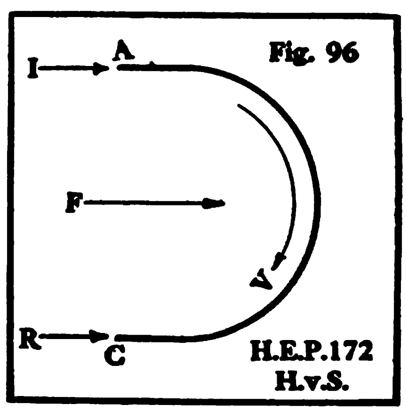


Fig. 96

H.E.P.172
H.v.S.

The theoretic work of the moving vane equals the product of the force exerted against its surface and the space traversed by the moving vane during a unit period of time, or, from Example Fig. 97,

$$k = F\,U = \frac{W}{g}\,(1 - \cos x)\,(V - U)\,U.$$

These deductions of the elementary characteristics of the force, velocities, and directions have been of surfaces moving in rectilinear directions, and, while not reflecting the exact conditions of the turbine, they gradually lead up to the practical utilization of these general principles by turbines in which the motion of the vane is rotary about a fixed point.

In Fig. 100 AC is a curved vane rotating around the fixed axis X; the stream enters at A;

r is the radius of rotation of the point of entry A;
U is the velocity of rotation or of revolution, its direction being normal to r;
V is the relative entry velocity;
A V V′ U is the entry parallelogram;
e, the angle formed by directions of flow and vane motion;
V′, the absolute velocity of the stream.

The water leaves the vane at C; r′ is the radius of this point;

U′ is the velocity of the vane at C, normal to r′;
V, the relative exit velocity of the water.

Completing the exit parallelogram C V, V″, U′,

x is the angle between relative exit velocity and direction of the vane motion reversed;
V″, the absolute exit velocity;
U′, the vane velocity; and
x′, the angle between absolute exit velocity and vane motion.

Heretofore, when the motion of the vanes was considered to be in rectilinear directions, the relative velocities of entry and exit water

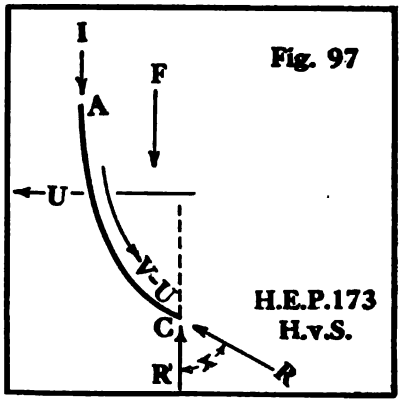


Fig. 97
I
F
A
U
V-U
C
R'
x
R
H.E.P.173
H.v.S.

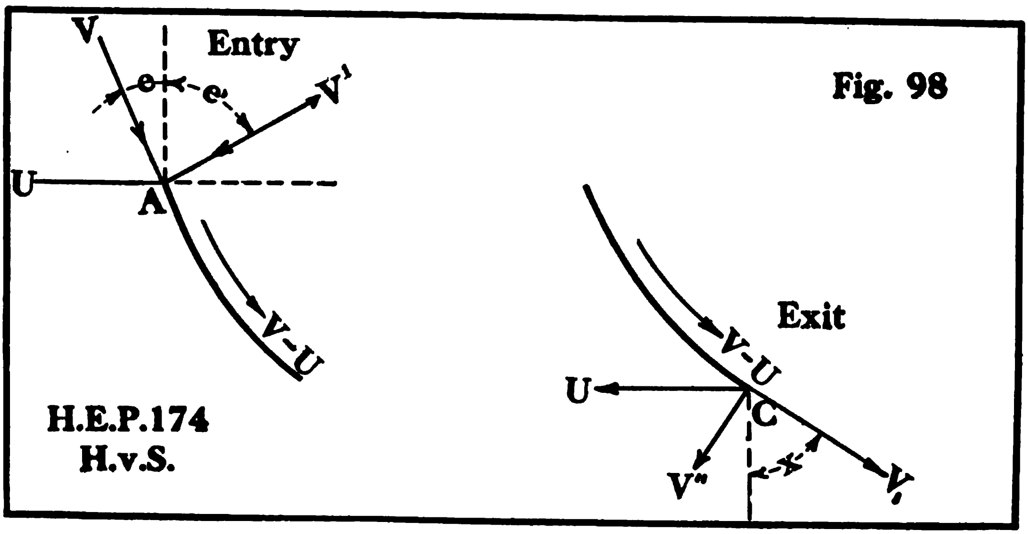


Fig. 98
V
Entry
e
e,
V'
U
A
V-U
Exit
V-U
U
C
V"
x
V,
H.E.P.174
H.v.S.

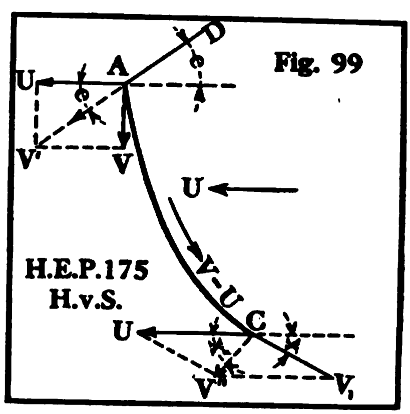


Fig. 99
D
A
U
e
V'
V
U
V-U
C
U
x
V"
V,
H.E.P.175
H.v.S.

were assumed to be equal, because, presumably, the vane's motion was equal at both ends; in the case now discussed, where the vane's motion is a rotary one equal, say, to n revolutions per minute,

$$U = 2 r \pi n \text{ and } U' = 2 r' \pi n \text{ or } \frac{U}{U'} = \frac{r}{r'}.$$

The impulse and reaction forces exerted on the vane, as due to the absolute entry and exit of the water on and off the vane, or from

$$F = \frac{W}{g} V' \text{ and } \frac{W}{g} V'', \text{ are}$$

$$\text{for the impulse } I = \frac{W}{g} V' \cos e \text{ and}$$

$$\text{reaction } R = \frac{W}{g} V'' \cos x',$$

and the work produced on the vane or k = impulse − reaction,

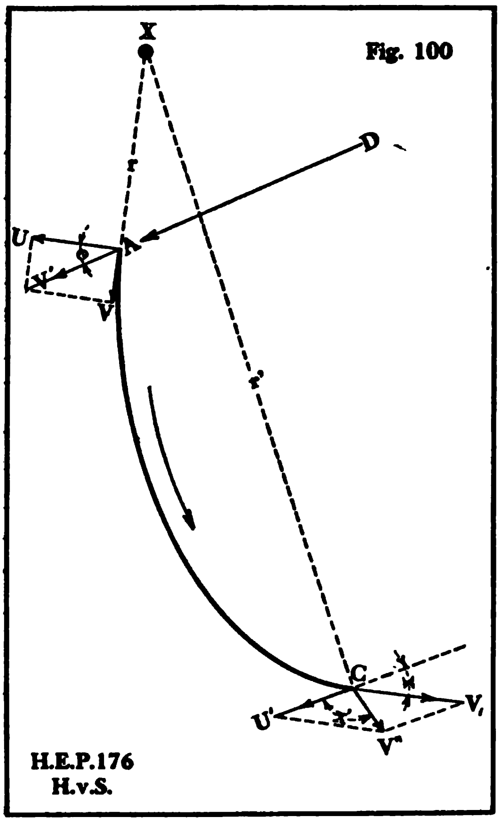
$$k = \frac{W}{g} (U V' \cos e - U' V'' \cos x').$$

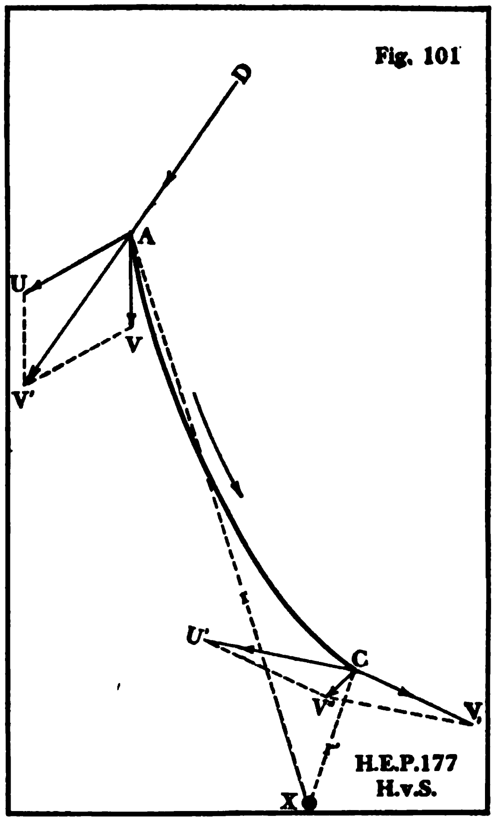
In Fig. 101 the vane has a rotary motion as in the last example, but the stream enters the vane at the extremity of its peripheral path and finds its exit nearer the axis, while these conditions of entry and exit were reversed in the previous example. These two represent inward and outward flow conditions as relating to the locus of the fixed axis around which the vane revolves; all the theories of one apply likewise to the other.

This completes the theory of hydraulic turbines as based upon the flow of water along the surfaces of moving vanes; as a matter of fact, the actual flow through the majority of turbines takes place in pipes rather than along surfaces, which, however, makes no difference in the theories expounded excepting that the static pressures are added and that the law of continuity of flow governs the water's progress while passing through the turbine.

Summarizing these theoretical fundamentals, it is found that the work of a turbine is caused by the dynamic energy of the water expressed by the product of its mass and motion acting through impulse and

reaction upon curved vanes of free rotary motion; that the degree of work thus imparted to the vane, or the wheel consisting of a system of such vanes, depends chiefly upon the relative values of the components of impulse and reaction force, which are influenced, if not entirely determined, by the designs of the vane or bucket, at the entry and exit points, whereby the velocities and directions of entry and exit flow are largely fixed; the





motion of the wheel or bucket becomes a factor in the work expression, while the shape of the vane between entry and exit is of lesser influence.

It is evident that turbine designs should aim at the ideal,—namely, *the greatest velocity of the moving vane with the highest value of work,*—and how this may be secured theoretically, and how far it is likely to be realized practically, under fixed conditions, will appear further on in this chapter.

ARTICLE 80. *Classifications of Turbines.*—Any mechanical device which whirls about a fixed point may be called a turbine. Applied to

hydraulic motors a turbine is a wheel which turns around a fixed shaft under the influence of flowing water by the force of impulse, pressure, and reaction, or a combination of these. Hydraulic motors are classified as water-wheels and turbines, which is descriptive of the older and modern types, the former comprising those which utilized the potential or the kinetic energy, while the turbine class includes all motors in which both of these forces unite in the production of the useful work. The water-wheel, in the above sense, need not be considered in connection with modern hydro-electric practice, as it is represented by the overshot, breast, and undershot wheels of the older mill practice which have been superseded by the modern turbine.

The primary classification of turbines is based upon the action of the water, which may be impulse, pressure, and reaction, but never weight only. It is by impulse when the water strikes buckets or vanes more or less perpendicularly and spreads out over the surfaces in all directions; by pressure when the water glides along the curved surfaces of vanes in one fixed direction, only partially filling the passages; and it is by reaction when water passes through closed passages formed by curved vanes in one fixed direction and completely fills these passages.

Turbines acting under pressure partake some of the impulse and some of the reaction characteristics, and therefore the classification with regard to the action of the water may be limited to impulse and reaction turbines, though even this is not absolutely correct, since a turbine of either class may, under certain conditions, partake of some of the characteristics of the other.

The theoretical differences between the impulse and the reaction turbines are expressed by the following paralleled conditions:

Impulse.	Reaction.
The flow is under atmospheric pressure through passages only partially filled;	The flow is free from atmospheric pressure through passages completely filled.
The velocity differs in all parts of the turbine without constancy of ratio;	The flow is subject to the law of continuity with a fixed ratio of velocity in all parts of the turbine.
The entry and exit being under atmospheric pressure, the energy at entry is kinetic only;	Entry and exit are under water and entry energy is potential.
The flow through the turbine is neither accelerated nor retarded;	The flow in passage is accelerated and retarded.
The effective head is that from upper pool level to the level of entry point;	The effective head is that from the upper pool level to the level of the point of exit.
The effective head cannot be made available to the level of the lower pool by means of draft tubes.	The head can be made available to the level of the lower pool by the aid of draft tubes within the limit of the atmospheric column.

The second important classification of turbines is based upon the direction of the entering and passing water as inward, outward, downward, or upward, or expressed in relation to the shaft as radial or axial or parallel, and for impulse wheels as tangential to the periphery of the wheel. Impulse turbines are therefore also called tangential turbines, while reaction turbines in this respect are defined as

Radial—outward flow, as in the Fourneyron type,

Axial—downward flow, as in the Jonval type,

Radial—inward flow, as in the Francis type,

Mixed flow, as represented by the modern American turbines.

A third turbine classification relates to the character of the control of the flow into the turbine by means of guide-wheels and gates, which are of cylinder, register, and wicket types, the details of which are explained further on; and the fourth and final classification is that based upon the method of installation as being on a vertical or a horizontal shaft.

Thus a turbine is properly designated as a vertical, wicket-gate, mixed-flow turbine, or a horizontal, cylinder-gate, radial inward flow turbine, or a tangential (impulse) turbine, which is always on a horizontal shaft.

Additional definitions to these quoted relate to manufacturer's special designs of shape, dimensions and spacing of buckets, gate devices, or particular structural features, or denoting the names of the originator of the type or of the maker.

ARTICLE 81. *Description of the Mixed-flow Reaction Turbine.*—A reaction turbine consists of the runner, guide-wheel, gates and rigging, and the case.

Fig. 102 shows a typical American reaction turbine runner, which, as compared with those used abroad, differs chiefly by the shape and dimensions of the buckets. The aim is to secure in the runner the largest possible discharge capacity with the smallest diameter and therefore the least quantity of metal, to utilize the largest ratio of the available reactionary force, and to obtain the highest speed of rotation.

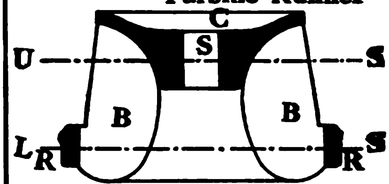
The velocity of the runner, under a given head, is inversely as its diameter; therefore, to increase the velocity by the reduction of the diameter would defeat the other equally important desideratum of a large discharge capacity; this latter is secured in the American type runner, at some advantage over others, by the shape of the buckets and

the axial depth of the entry passages. These entry areas are made as large as possible not by an increase of the runner's diameter, but by deepening the vertical sections of the vanes, while the outflow sections

Fig. 103

H.E.P.179
H. v. S.

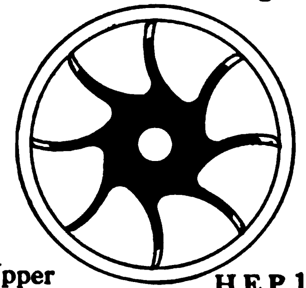
Fig. 104
Typical Reaction
Turbine Runner



Axial Section H.E.P. 180
H.v.S.

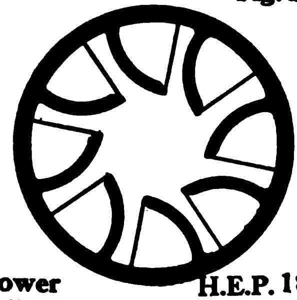
are enlarged by the broad dishing of the vanes. In this manner the runner is given, as compared to its diameter, an abnormally large entry and exit area, while the curvature of the vanes is such that the flow, entering inward radially, passes downward axially and leaves outward

Fig.105



Upper
radial Section H.E.P.181
H.v.S.

Fig.106



Lower
radial Section H.E.P. 182
H.v.S.

radially, and therefore its exit direction is nearly reversed from that of entry, which, as has been shown in Article 79, Fig. 96, results in the utilization of the greatest amount of the available reaction force.

Fig. 103 shows the runner in the upright position; E is the entry section, and the arrows e x indicate the reversal of the flow direction between entry and exit.

Fig. 102

H.E.P.178
H. v. S.

Reaction Runner.

Fig. 107

H.E.P.183
H. v. S.

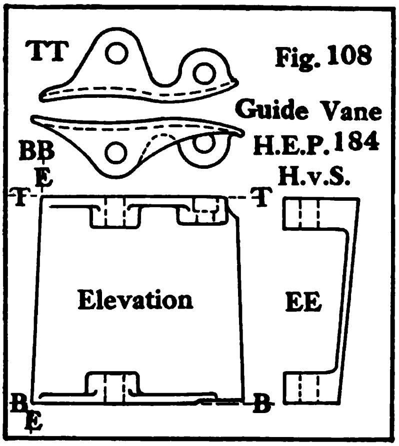
Wicket-Gate Guide-Wheel.

Fig. 104 shows an axial section of the runner and its parts, H being the hub, C being the crown, R being the rim, B being the bucket.

Fig. 105 is a radial section just below the crown plate, showing the shape of the top ends of the vanes, their connection to the hub, and the boring left for the shaft.

Fig. 106 is a radial section near the bottom of the runner through the rim band where the dishing out of the vanes first begins; the metal here is no thicker than at the upper section shown in Fig. 105.

Runners are usually made of one cast with the hub; only a few of the manufacturers in this country depart from this practice by casting all the vanes separately and then setting them in the mould of the hub cast and thus securing their fixed position around it.



Guide-wheels are designed to suit the different gate devices; they generally consist of two circular plates between which the *guide-vanes* are fixed stationary in positions diagonal to the axis of the wheel; this guide-wheel slides over the crown-plate and rim band of the runner.

Fig. 107 shows a guide-wheel in which the guide-vanes are the *gates;* it represents the general type of the American *wicket gate,* the movable shutters or gates performing also the office of guiding the water into the runner. The operation of the shutters is by means of a segmental spur gear, G, actuating the gate rods R, the ends of which are firmly secured to the bottom of the gear section.

Fig. 108 shows horizontal and vertical sections of the guide-vanes and shutters, the upper at TT being taken at the top of the shutter and BB at the bottom, while EE is a vertical cross section.

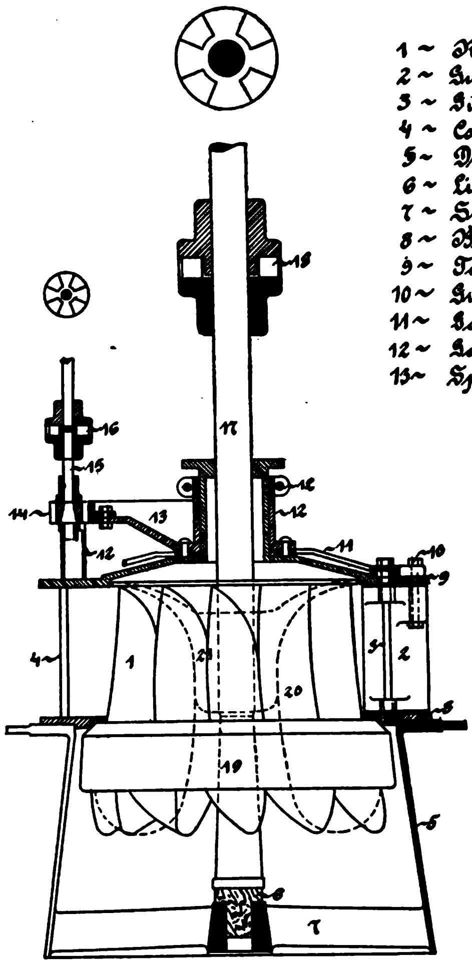
The guide-wheel with fixed guide-vanes is employed in connection with the other gate devices; cylinder and register type; the former consists of a cylinder sliding over the outside or inside periphery of the guide-wheel frame, while the second, as its name implies, resembles a hot-air register, being arranged in the crown-plate or periphery of the guide-wheel.

Fig. 109 is the complete plan of an American, mixed-flow, wicket-gate turbine, in which

1 represents the runner proper;
2 represents the guide-vanes and gates;
3 represents the guide bolts securing the gates to the guide or gate-wheel frame;
4 represents the column bolts which connect the circular plates of the guide or gate-wheel;
5 represents the draft tube secured to the bottom plate of the gate-wheel;
6 represents the lignum vitæ packing which surrounds the step of the turbine shaft;
7 represents the spider or frame in which the turbine shaft rests or is stepped;
8 represents the bottom plate of the gate-wheel;
9 represents the top plate of the gate-wheel;
10 represents the guide-pins;
11 represents the gate-rods by which the gates are operated;
12 represents the gate-operating device;
13 represents the spur-gear sector to which the gate-rods are secured;
14 represents the spur pinion which is operated by means of the shaft;
15 represents the gate shaft which is in practice connected to the turbine governor;
16 represents the coupling of the gate shaft to its upper or lateral connection, as the turbine may be placed on a vertical or horizontal shaft;
17 represents the turbine or wheel shaft;
18 represents the turbine or wheel shaft coupling;
19 represents the rim or runner band;
20 represents the runner bucket;
21 represents the runner hub.

Fig. 110 shows the complete plan of an American, mixed-flow, cylinder-gate turbine, in which the following are the detail parts and their nomenclature:

1 represents the runner;
2 represents the runner hub;



List of Parts.

1 ~ Runner
2 ~ Guide Vanes
3 ~ Guide Bolts
4 ~ Column Bolts
5 ~ Draft Tube
6 ~ Lignum Vitae Step
7 ~ Spider
8 ~ Bottom Plate
9 ~ Top Plate
10 ~ Guide Pin
11 ~ Gate Rods
12 ~ Gate operating Device support
13 ~ Spur Gear Sector
14 ~ Spur Pinion
15 ~ Gate Shaft
16 ~ Coupling for Gate Shaft
17 ~ Wheel Shaft
18 ~ Coupling for Wheel Shaft
19 ~ Runner band
20 ~ " bucket
21 ~ " hub

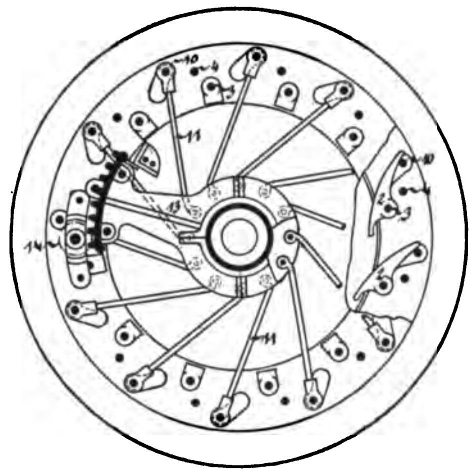
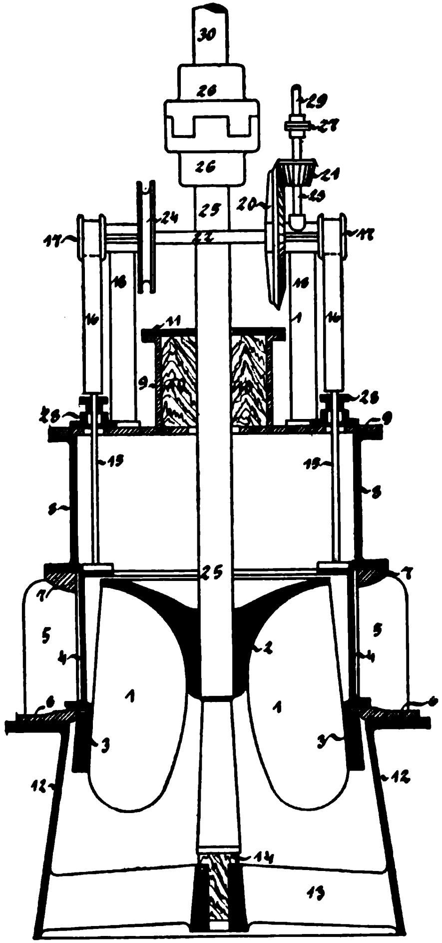


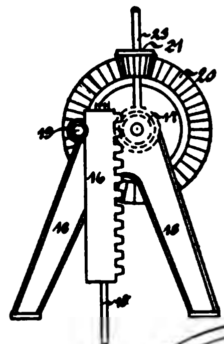
Fig. 109
American
Reaction Turbine
low & medium head
wicket gate

H.E.P. 185
H.v.S.

3 represents the runner rim band;
4 represents the cylinder-gate operating; in this case, on the inside of the gate-wheel;
5 represents the guide-vanes;
6 represents the bottom plate or ring of the guide-wheel;
7 represents the top plate;
8 represents the dome plate of the guide-wheel;
9 represents the box or lid containing lignum vitæ bearing;
10 represents the lignum vitæ blocks;
11 represents the lid or top cover of the bearing;
12 represents the draft tube;
13 represents the spider;
14 represents the lignum vitæ step;
15 represents the gate-rods connecting the cylinder gate to the upper operating device;
16 represents the spur racks;
17 represents the spur pinions by which the movement of the cylinder gate is controlled;
18 represents the brackets to which the gate rigging is secured;
19 represents the roller, shown in the horizontal section, confining the spur rack to the spur-gear pinion;
20 represents the bevel gear;
21 represents the bevel pinion actuating the spur pinion and rack;
22 represents the horizontal gate shaft;
23 represents the vertical gate shaft;
24 represents the cable wheel;
25 represents the turbine or wheel shaft;
26 represents the turbine or wheel-shaft coupling;
27 represents the vertical gate-shaft coupling;
28 represents the vertical gate-shaft stuffing-box located below the spur rack;
29 represents the vertical gate-shaft extension finally connected to the turbine governor or hand wheel;
30 represents the turbine-shaft extension.

The design of the *housing or casing* of the turbine depends upon the method of operation. When the water is supplied to the turbine by a feed pipe, the casing consists of a shell completely enclosing the turbine; when the turbine is placed in an open bay, the case or draft chest encloses





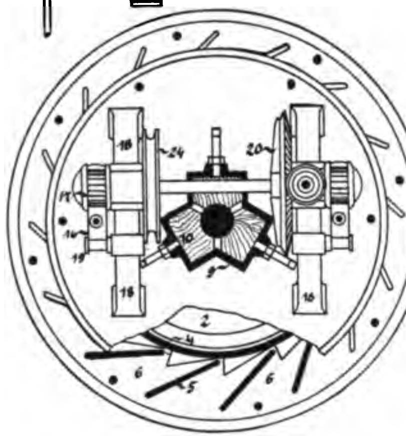


Fig. 110
American
Reaction Turbine
low or medium head
Cylinder Gate

List of Parts.

1 ~ Runner Buckets
2 ~ " Hub
3 ~ " Band
4 ~ Cylinder Gate
5 ~ Guide Vanes
6 ~ " " Bottom Ring
7 ~ " " Top "
8 ~ Dome
9 ~ " Lid with Lignum Vitae Bearing
10 ~ Lignum Vitae Blocks
11 ~ Lid.
12 ~ Draft Tube
13 ~ Spider
14 ~ Lignum Vitae Step
15 ~ Gate Rods
16 ~ Spur Racks
17 ~ " Pinions
18 ~ Brackets
19 ~ Roller
20 ~ Bevel Gear
21 ~ " Pinion
22 ~ Horizontal Gate Shaft
23 ~ Vertical " "
24 ~ Cable Wheel
25 ~ Wheel Shaft
26 ~ " " Coupling
27 ~ Gate " "
28 ~ " " Stuffing Boxes
29 ~ " " Extension
30 ~ Wheel " "

H.E.P. 186
H.v.S.

all but the guide-wheel passages through which the water finds its way into the runner.

Fig. 111 shows such a draft chest for a double turbine; it is a cast of two parts with a man-hole in the top or dome. The turbine runners are inserted at the ends EE and the case is secured to the floor structure by its bed flange F.

Fig. 112 shows the turbines, runners, and gate-wheels assembled and connected by one shaft, all ready to be placed in the casing or draft chest shown in Fig. 111.

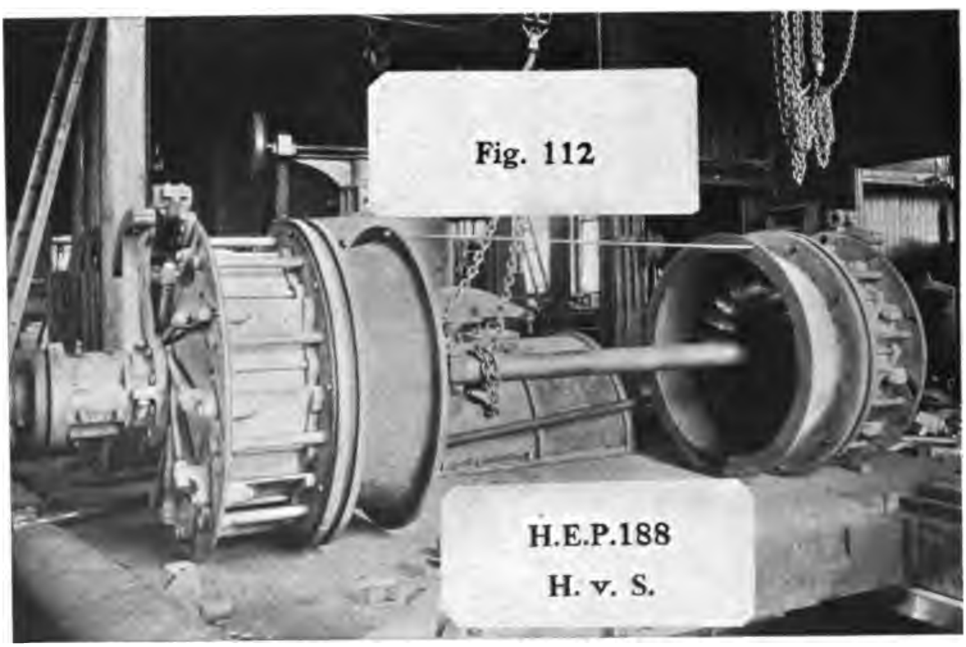
ARTICLE 82. *Description of a Central-discharge Reaction Turbine.*—This type is of the general class of Fourneyron turbines; the runner is very shallow, as compared with that of the mixed-flow turbine; the guide passages are of little depth and the guide-vanes are curved; the water enters simultaneously into all guide passages from a scroll-shaped supply pipe which encircles the runner and has openings along its interior periphery through which the water spouts into the guide passages; this supply pipe gradually decreases in area, so that the velocity of the entering water is constant at all points of entry. This special supply feature has given to these turbines the French name of volute.

These turbines are especially adapted to high-entry velocities and therefore high heads; such plants as those at Trenton Falls, Niagara, and Shawinigan Falls are equipped with them; the largest turbine yet constructed operates at the latter plant and is of this type with a capacity of 10,500 horse-power.

Fig. 113 gives a complete plan of this type of turbines as they are constructed in this country.

No. 1 represents the runner;
No. 2 represents the guide-vanes;
No. 3 represents the guide-vane plates;
No. 4 represents the supply chamber passing around the runner;
No. 5 represents the crown plate of the turbine casing;
No. 6 represents the elbow of the discharge pipe;
No. 7 represents the draft-tube ring;
No. 8 represents the main-shaft bearing;
No. 9 represents the thrust bearing;
No. 10 represents the pedestals;
No. 11 represents the shaft coupling;
No. 12 represents the main shaft;

Double Turbine Draft Chest.



Assembled Twin Turbines.

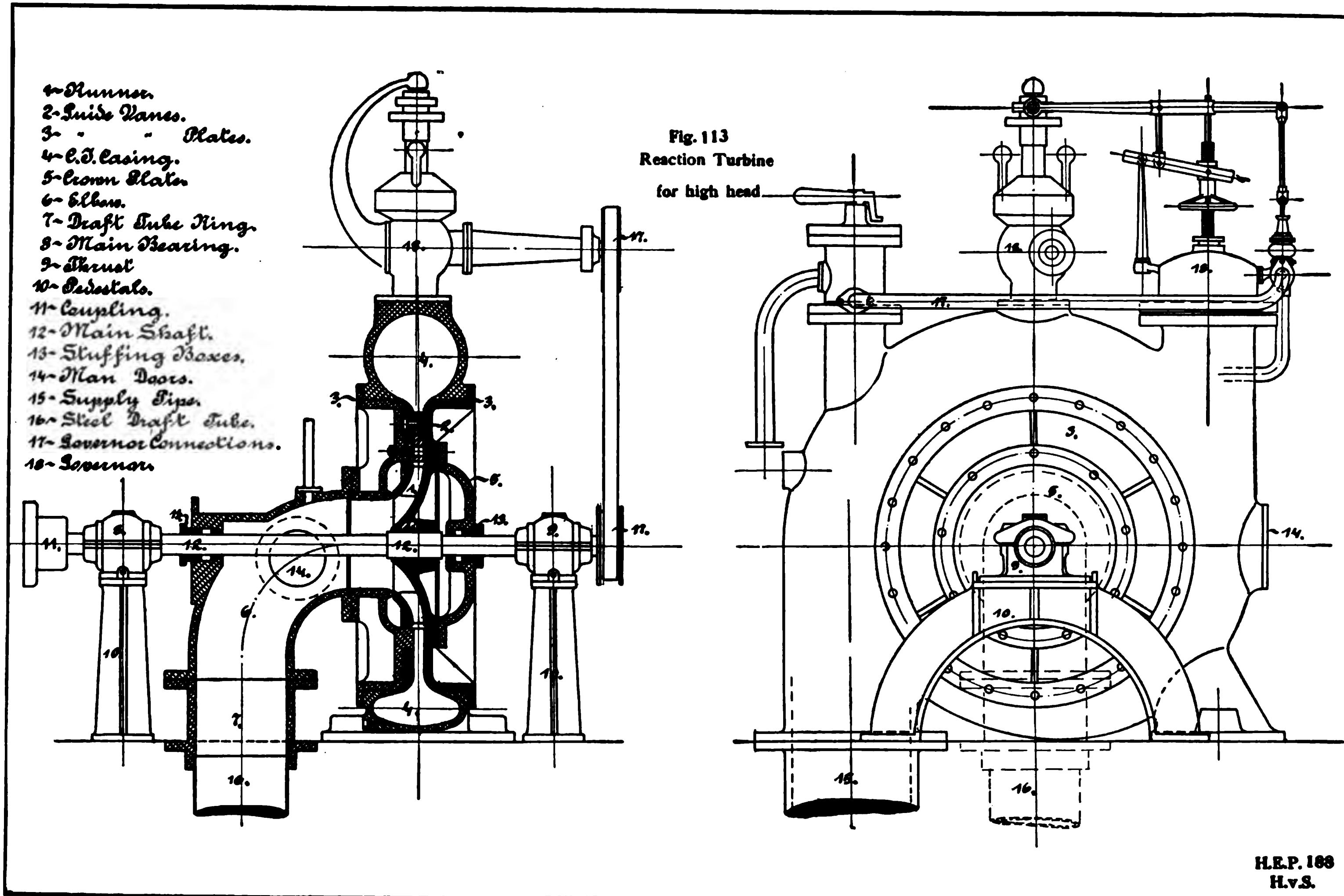


Fig. 113
Reaction Turbine
for high head

No. 13 represents the shaft stuffing-boxes;
No. 14 represents the man doors in the case;
No. 15 represents the supply pipe;
No. 16 represents the draft tube;
No. 17 represents the governor connections;
No. 18 represents the governor.

ARTICLE 83. *Description of an American Impulse Turbine.*—In this country the impulse turbines, as a distinct type, are represented by that shown in Fig. 114, consisting of a wheel or disk which carries on its periphery cup-shaped buckets, the whole being encased and operating on a horizontal shaft. The buckets are of double cups, the central partition splitting the striking jet in equal parts as to volume. The water spreads out all over the bucket surfaces and is deflected 180° from the entry direction when it drops from the buckets. This class of turbines is especially adapted to the highest heads and is commonly called the hurdy-gurdy wheel.

In the figure showing a plan of it

No. 1 represents the wheel disk;
No. 2 represents the wheel buckets;
No. 3 represents the wheel shaft;
No. 4 represents the ring oil bearings;
No. 5 represents the base plates;
No. 6 represents the nozzle;
No. 7 represents the gate valve;
No. 8 represents the supply pipe;
No. 9 represents the hand-wheel stand;
No. 10 represents the hand-wheel;
No. 11 represents the wheel case;
No. 12 represents the floor plate.

ARTICLE 84. *Theory of the Draft Tube.*—The draft tube is an air-tight cylindrical extension secured to the lower or discharge end of the runner. The theory of its service is based upon the atmospheric pressure of 14.72 pounds per square inch, which equals the weight of a column of water about 34 feet high; in other words, a column of water of this height, and *at rest,* is balanced and therefore held in equilibrium by the atmospheric pressure. When the water in this column is in motion, falls, the head represented by the velocity with which it falls at the point of exit from the lower end of the column must also be balanced by the

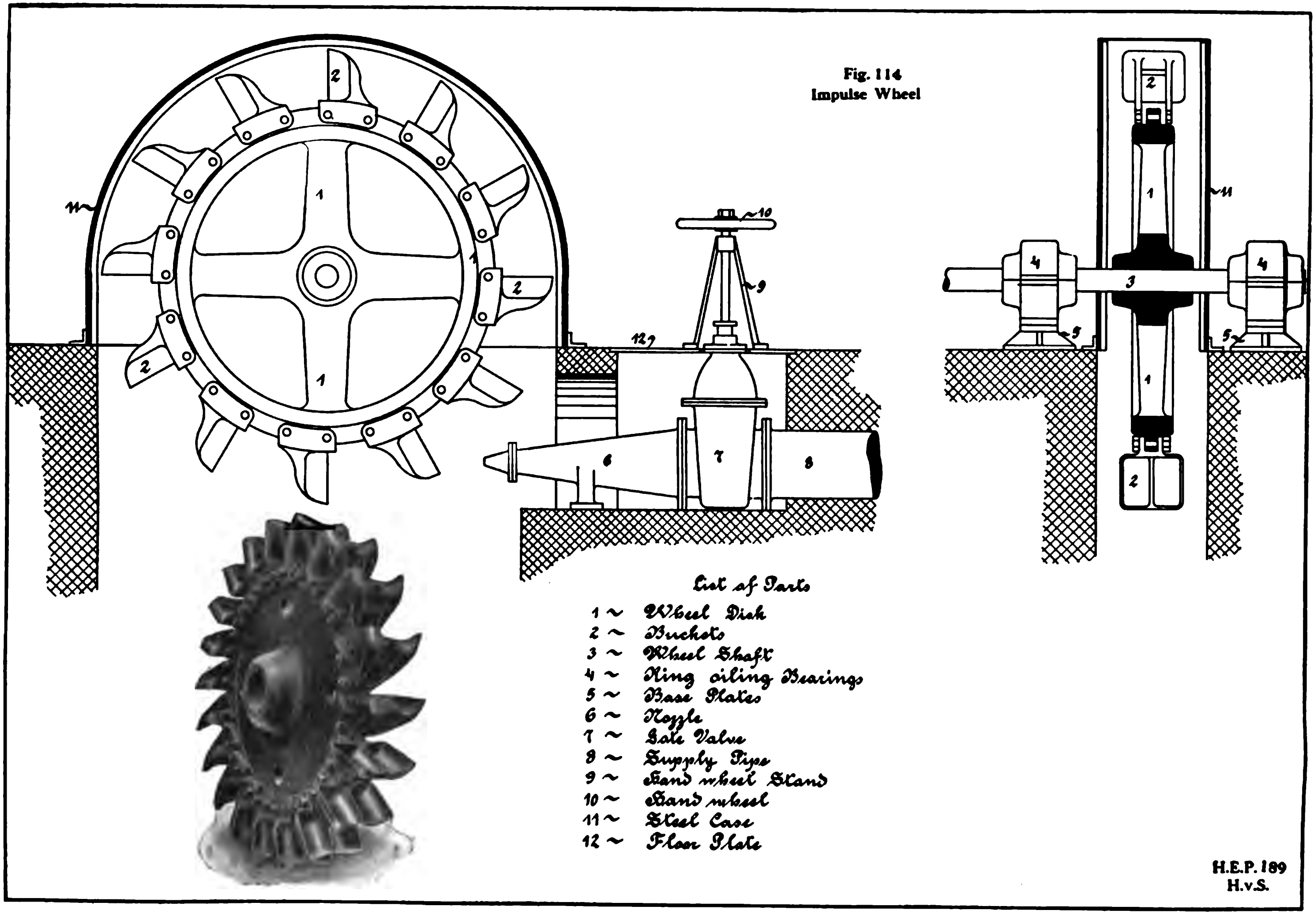


Fig. 114
Impulse Wheel

atmospheric pressure, and the theoretic height of the column held in equilibrium, or which can be maintained in a draft tube, is then reduced by this head.

If the exit velocity from a turbine runner is that due to a head of thirty feet, or approximately $0.2\sqrt{2gh} = 7.2$ ft., and the water continues with this velocity to the exit end of the draft tube, the head represented by this velocity is $h = 7.2^2 \div 2g = 0.8$ ft., and the theoretic height of the column held in equilibrium in the draft tube is 34 — 0.8.

If the velocity in the draft tube were 46.8 ft. per second, then the corresponding head would be $h = 46.7^2 \div 64.4 = 34$ ft. (about), and the theoretic height of the column balanced by the atmospheric pressure would be zero.

Any other losses of head occurring during the water's passage through the draft tube, as well as that represented by the energy remaining in the finally escaping water, must be deducted from the theoretical 34 feet.

From the foregoing it is apparent that by the use of the draft tube the exit velocity from the runner may be materially reduced during its passage through the draft tube by gradually increasing the flow area in the latter, and that therefore otherwise lost head is conserved and available to produce useful work;

That the opportunity of gain from this source is greater with a long than a short draft tube;

That the use of the draft tube permits the placing of the turbine at a convenient height above the tail water, while without it the turbine must be placed at or below the lower level;

That the draft tube makes it feasible to place the turbine on a horizontal shaft at sufficient height above the lower pool to allow of directly connecting it to the electric generator or other machinery; and therefore it is true that the draft tube brought out the horizontal turbines.

It will also be understood from the theory presented that draft tubes cannot be used, as such, with impulse turbines, since the water column passing through the impulse turbine is not continuous.

The practical application of the draft tube, its most efficient design and length under differing conditions, is fully treated in a succeeding article.

ARTICLE 85. *Theory of Deducting Turbine Efficiencies.*—The useful work represented by the turbine output is the total available energy less

the losses occurring in the turbine; these losses are of three kinds, hydraulic, mechanical, and un-utilized residues. The first are of manifold origin, generally caused by friction, impact, and leakage; they are best identified by tracing the water's flow as it passes through the turbine.

(a) *In passing through the guide-vanes* some loss is experienced due to the friction of the passing water against the walls of vanes; the best conditions exist when the walls of the vanes are parallel, thus avoiding contraction of the passing vein, and when the guide-vanes are made of hard metal and their surfaces are polished. It is especially important that the guide-vanes be frequently examined and repolished, as their surfaces are rapidly roughened, which is the common experience when the water carries considerable sand and other suspended matter.

(b) *When passing from the guide passages into the runner* the probable losses are due to impact, to retardation, and to leakage through the clearance between the guide-wheel and the runner. If the water, upon entering the runner buckets, strikes the vanes tangentially, the only other cause of loss from impact is the striking of the edges of the buckets as they pass by the guide openings; of course this can not be avoided, but the effect may be minimized by reducing these bucket-vane edges to the sharpest practical shape and thus maintaining them. Some clearance must be left between the guide-wheel and the runner, as the former is stationary while the latter rotates, and therefore leakage will take place, particularly as the interior water pressure exceeds the exterior; the clearance should be a practical minimum, which necessitates accurate and true machining of the runner bands and guide-wheel plates. This loss from leakage through this clearance is one which increases with wear and cannot well be guarded against.

(c) *Passing through the runner* the loss is chiefly that due to the friction against the walls of the bucket vanes, which should be of hard material and polished; in fact the conditions here are analogous to the passage through the guide-vanes.

(d) *Passing out of the runner* the principal loss is caused by the change in the velocity, which may be kept at a minimum by the proper designing of the draft tube.

(e) *Passing through the draft tube* losses are caused by friction against the tube's walls, which may be considerable, or very small, depending solely upon the proper design and construction of the tube with a view of avoiding joint, rivet, and other obstructions and maintaining the surfaces as smooth as practicable by frequently coating them.

(f) *Passing out of the draft tube* losses may be caused by obstructions to the free escape of the water; the water cushion below the draft tube may be too shallow, the tail-flume dimensions insufficient or badly distributed.

These six may be summed as the hydraulic losses or inefficiencies; their aggregate depends upon the conditions pointed out; when design and construction are the most suitable for the purpose and the best, the

hydraulic losses may be taken to aggregate from 10 to 12 per cent., while they may be double of this where design is faulty or construction and finish are indifferent.

The losses due *to mechanical causes* are chiefly those of friction in shaft bearings, which with the best available appliances may be taken for each bearing at from 1 to 2 per cent., while it may be double or much greater when the true alignment of the shaft is permitted to be disturbed.

The last of the losses heretofore enumerated is that represented by the *unutilized energy* remaining in the escaping water; proper design should keep the exit velocity to 0.2 or 0.25 of the velocity of total available head, and beyond this no further reduction of this loss, with a proper use of the draft tube, can be secured. This last loss from unutilized energy will be from 5 to 6 per cent.

Summarizing these losses in reaction turbines:

hydraulic losses	11 to 12 per cent.
mechanical losses for two bearings	2 to 4 per cent.
unutilized	5 to 6 per cent.
	18 to 22 per cent.

representing the best theoretic conditions, and therefore obtainable efficiencies, of output of from 78 to 82 per cent.

It will be noted that two of the causes of losses grow out of the use of the draft tube, and these would not occur were no draft tube employed; the significance of this applies only to impulse turbines in which, for this and other reasons, a considerably higher efficiency of output can be realized, frequently reaching 90 per cent. and more.

ARTICLE 86. *Typical Turbine Installations.*—Reaction turbines may be installed

(a) On vertical shaft,
(b) On horizontal shaft,
(c) Cased and supplied through penstock,
(d) Drowned in open bay,
(e) Cased in pairs,
(f) Drowned in tandem,
(g) Drowned side by side,
(h) Drowned superposed.

Impulse turbines are always placed in cases and operated on a horizontal shaft.

The different installations of the reaction turbine are illustrated by succeeding figures, and the principal dimensions, such as are required in planning the power house, are given on diagrams; no particular make of turbine is herein referred to, but the information in these figures and diagrams applies generally to the reaction turbines manufactured in this country and may be safely accepted for the purpose above indicated.

Fig. 115 illustrates the *vertical turbine drowned*. The unit of this installation may consist of one or of several wheels; the generator may be coupled to the vertical shaft, a practice which is quite general abroad, in which case the generator operates horizontally and is sometimes called the umbrella type. The installation of the American Niagara plant is of this arrangement. More frequently the vertical turbine shaft is geared to a driving shaft, the generator being coupled to the latter, and in that case several turbines may be thus installed in one power unit, all being geared to a union shaft, so that each can be cut out separately. The turbines of the same unit may be placed in one or separate bays, the latter arrangement allowing of making repairs to any turbine of the multi-unit without stopping the operation of the remaining wheels.

The installation of vertical turbines drowned and geared to a driving shaft represents the oldest, the mill-power, practice in the utilization of water-power; it is also frequently chosen for hydro-electric plants, though it does not yield high output efficiency. Vertical turbines drowned, with generators coupled to the turbine shaft, constitute the latest developed installation, which is capable of yielding the highest obtainable output efficiency of reaction turbines.

This installation is suitable for the lowest and for high heads, the limitations being the length of shaft, the weight of shaft, and the cost of open-bay construction; the Niagara installation represents the high-head limit, to the present time, being 175 feet; the turbines, however, are not strictly of the type here illustrated, but are double Francis turbines. The lowest head utilized by a hydro-electric plant, to the author's knowledge, is that at Rechtenstein, Austria, being a trifle less than five feet; the turbines are vertical and drowned, the turbine shaft being geared to the driving shaft. This arrangement is peculiarly adapted to very low heads, on account of the smaller depth of the turbine compared to its diameter.

The power-house design suited for this installation is described in Article 76 and illustrated in Figs. 84 and 88. The guide-wheel rests upon

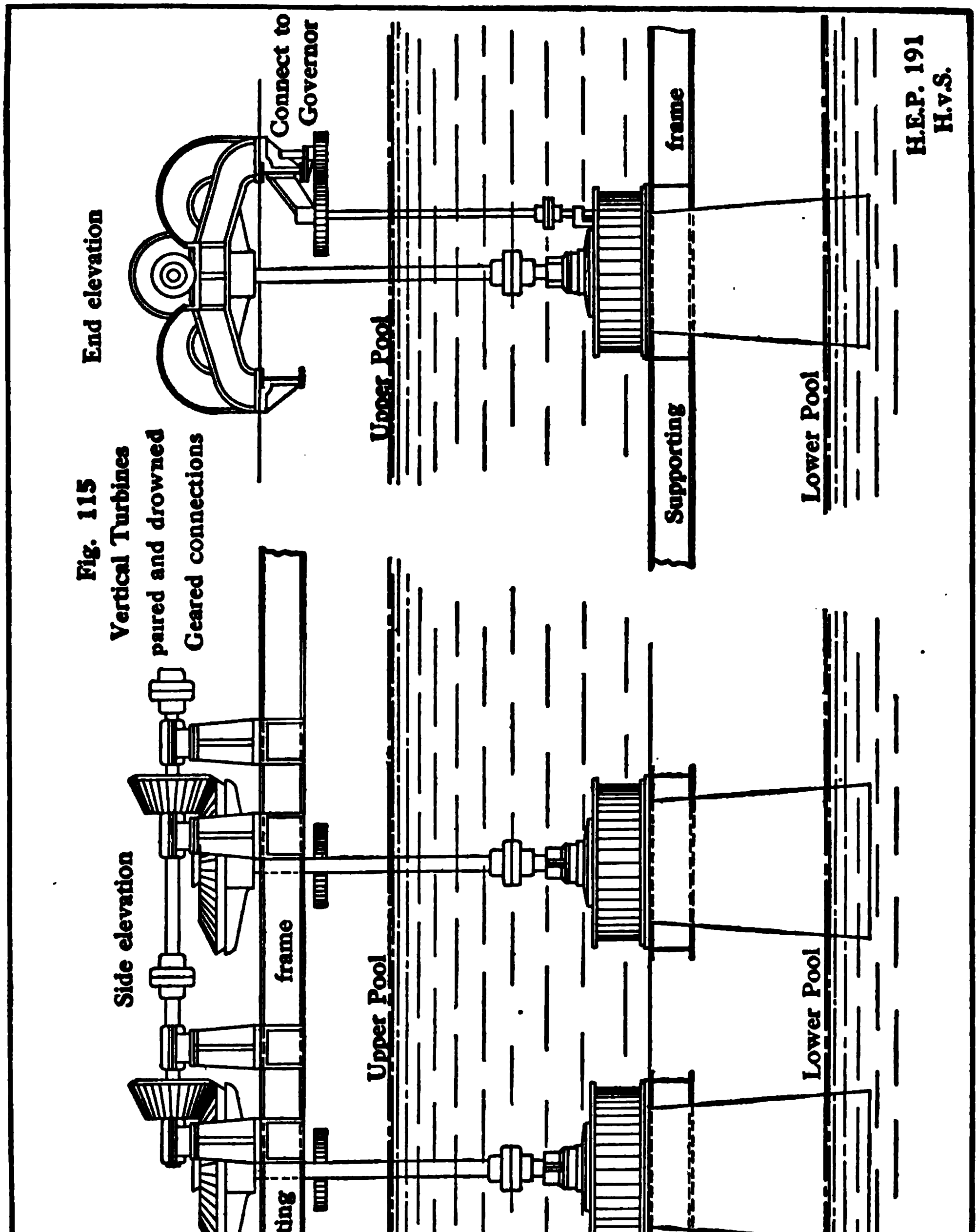
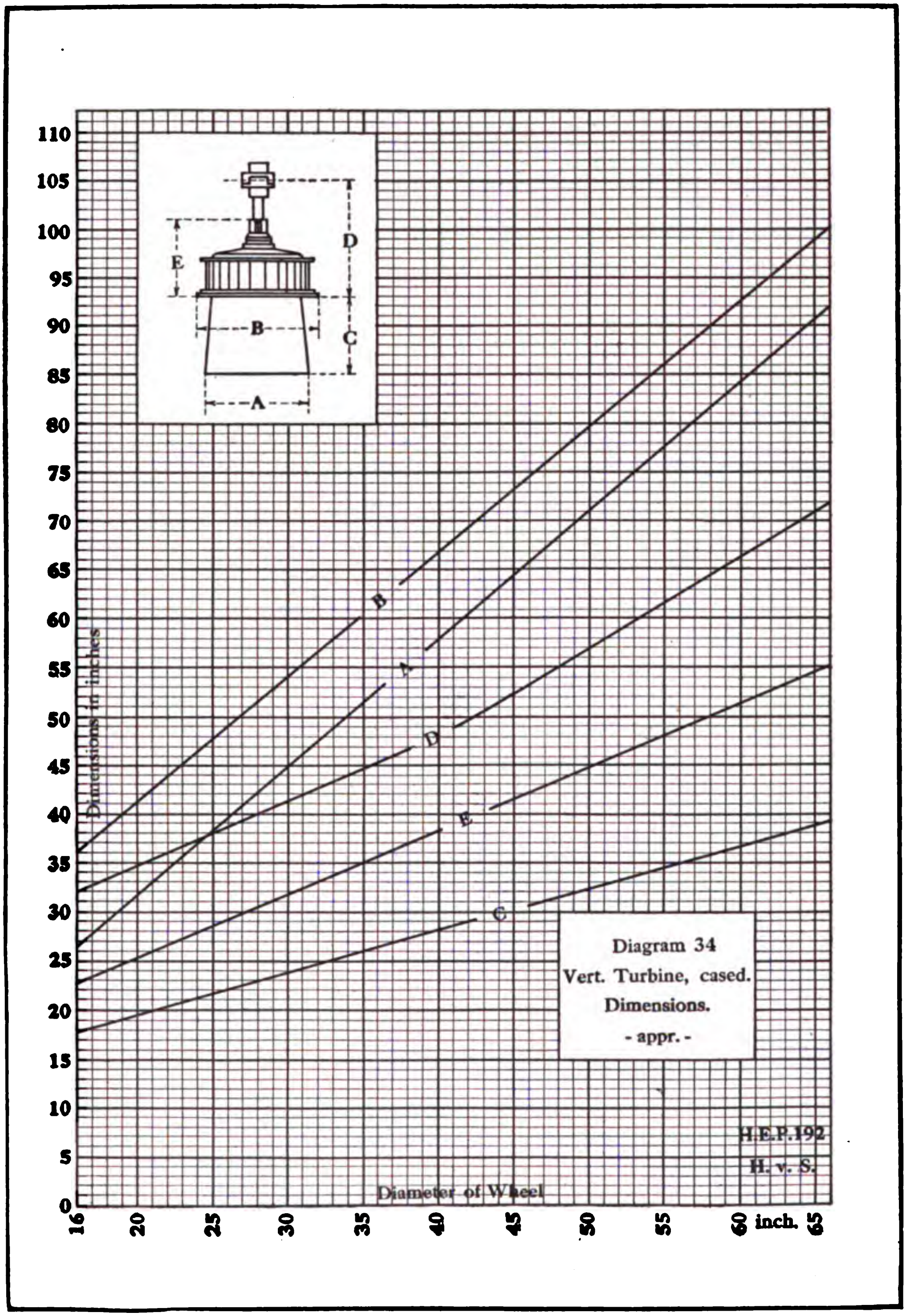


Fig. 115
Vertical Turbines
paured and drowned
Geared connections


110
105
100
95
90
85
80
75
70
65
60
55
50
45
40
35
30
25
20
15
10
5
0
Dimensions in inches
D
E
B
C
A
B
A
D
E
C
Diagram 34
Vert. Turbine, cased.
Dimensions.
- appr. -
H.E.P.192
H. v. S.
Diameter of Wheel
16
20
25
30
35
40
45
50
55
60
inch.
65

supporting frames of steel members secured to the pit structure; the gearing is similarly placed on an upper floor.

Diagram 34 gives the approximate exterior dimensions of a single vertical turbine drowned.

Example.—For a 40″ turbine the height from the supporting frame to the shaft coupling, D in the diagram, is 48″; the diameter of the guide-wheel, B in the diagram, is 67″; the total height from the top of the thrust bearing to the lower end of the draft tube, E + C in the diagram, is 66″; the required width of the open bay in which this turbine is placed is 2 B or 11′ 2″; the length of the bay for two such turbines is double the width or 22′ 4″.

In this manner the dimensions of the turbine bay for this installation, and therefore of the power house, can be readily determined from this diagram.

In this category of turbine installations falls that of serial turbines, illustrated in Fig. 87 of Article 76, which is specially adapted to fluctuations of head; in this, vertical turbines drowned are placed one above another, a separate tail-flume being provided for each, by which arrangement they are available to operate singly or jointly; this represents the only method at present known which solves the problem of utilizing excessive head fluctuations, and is deserving of far more attention than is now given it in the American practice.

Fig. 116 shows the installation of *vertical turbines cased*, the water being supplied through penstocks. The cases are placed upon and secured to supporting frames, the penstock or supply pipe entering the case at the side or top. The shaft connection and generator drive are similar to those described for the vertical turbine drowned. The determination between the drowned and cased installation of vertical turbines must be based upon the respective cost of open-bay construction and of turbine casing and penstocks; operation and output efficiencies are alike in both. This installation offers no special advantages with low heads; it is frequently met with in mill-power plants, the turbines being placed un-housed over the tail-race and geared to the driving shaft of the mill.

The dimensions of vertical turbine cases are, generally speaking, the same as those given for the open bay of drowned turbines.

Fig. 117 illustrates the installation of a *pair of horizontal turbines drowned.*

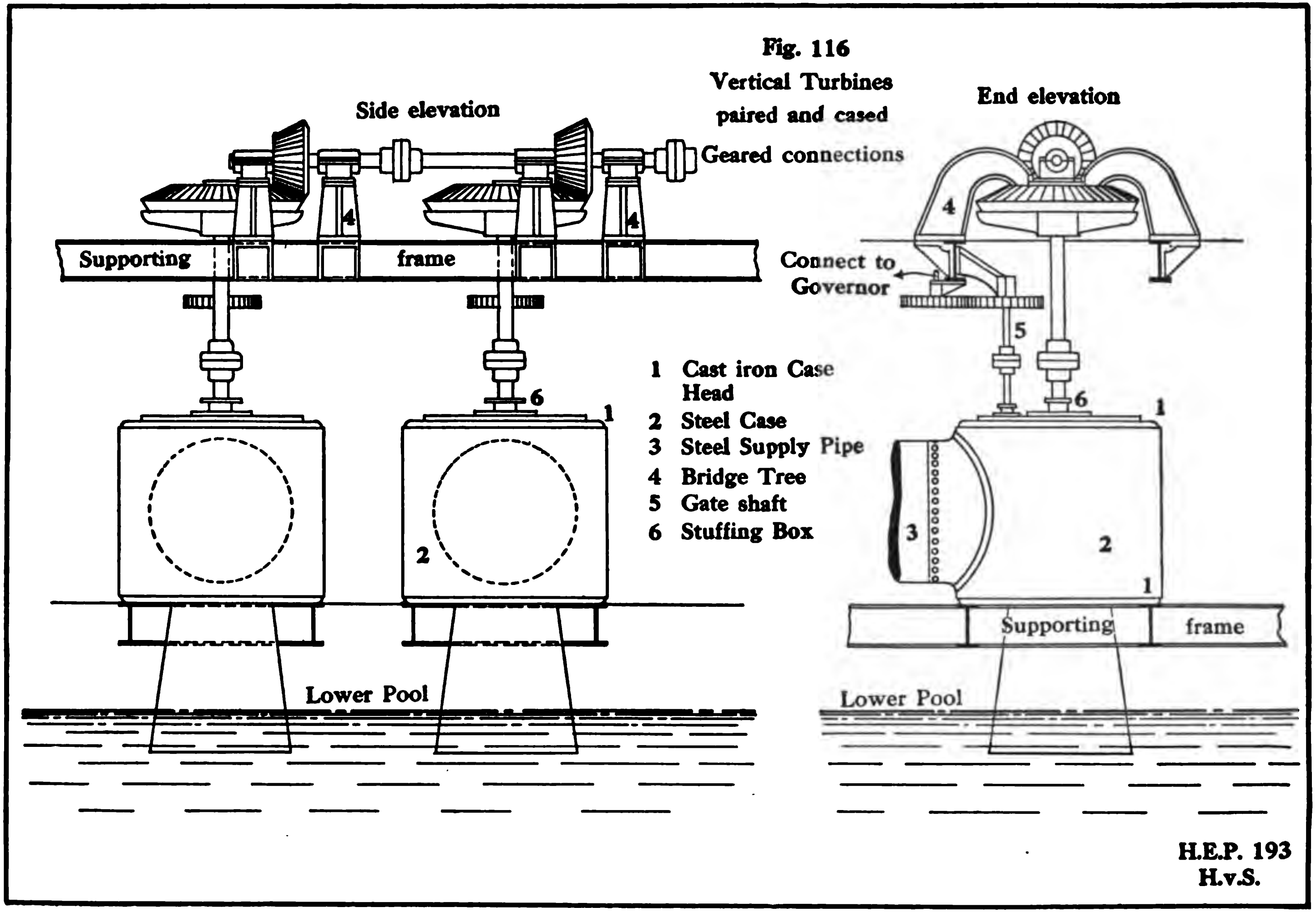


Fig. 116
Vertical Turbines
paired and cased

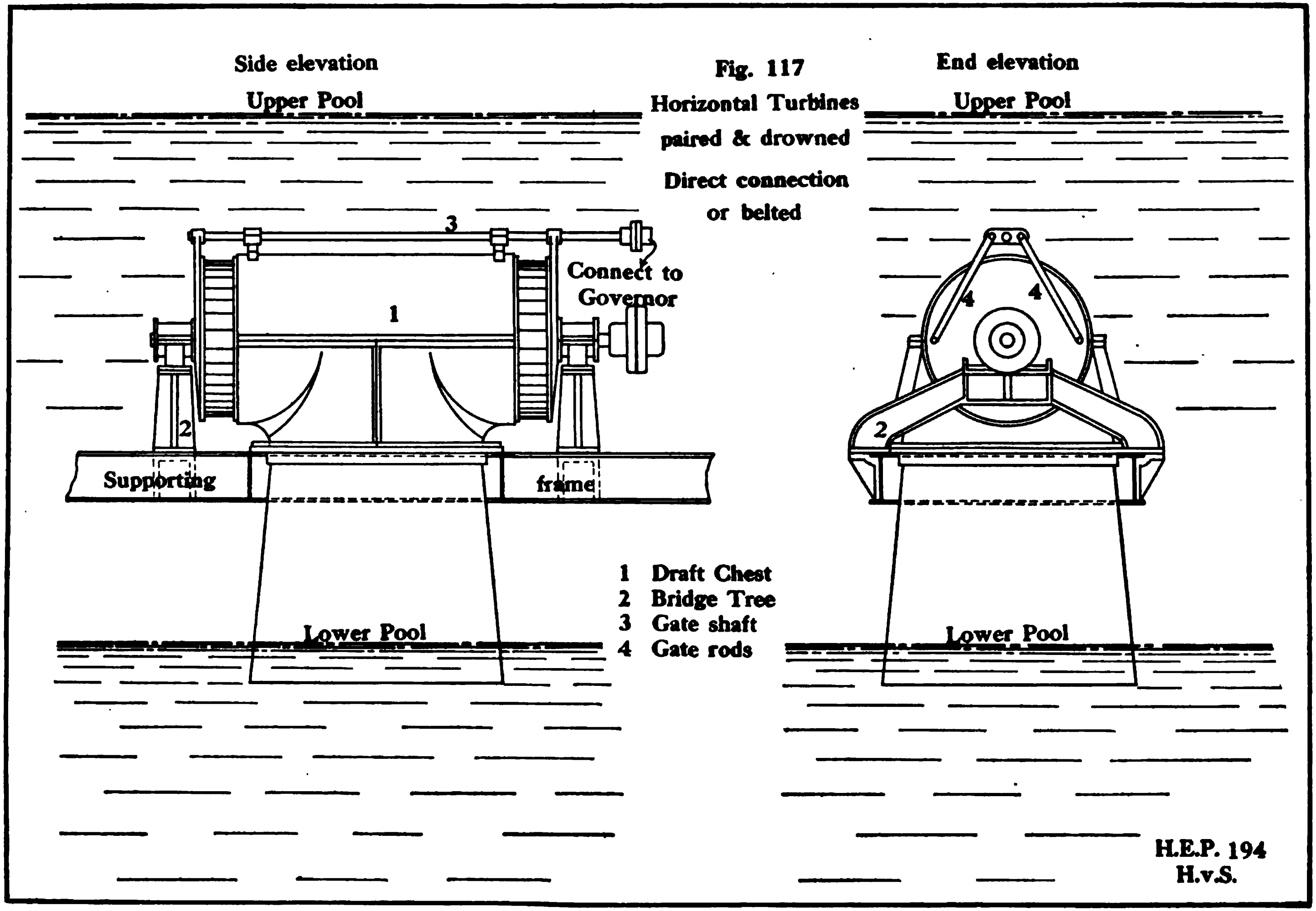


Fig. 117
Horizontal Turbines
paired & drowned
Direct connection
or belted

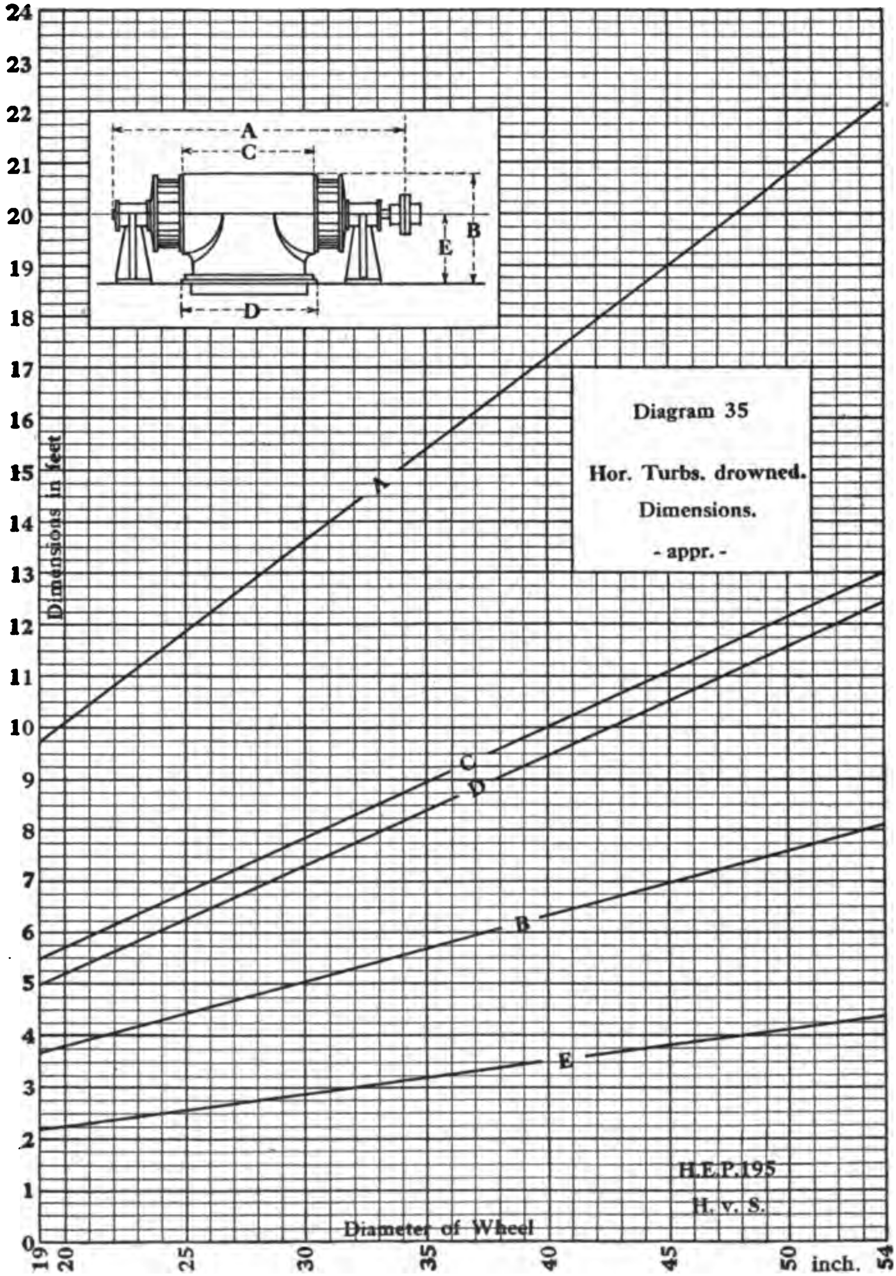


Diagram 35
Hor. Turbs. drowned.
Dimensions.
- appr. -
A
B
C
D
E
Dimensions in feet
Diameter of Wheel
inch.
H.E.P.195
H. v. S.
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
21
22
23
24
19
20
25
30
35
40
45
50
54

At the outset it may be stated that this represents that type of turbine installation best adapted to low and medium heads and is, next to the vertical drowned and umbrella generator plant, of highest practical efficiency.

The turbine runners are secured to the ends of a union draft chest, as was shown in Fig. 112 of Article 81, both connected to one shaft, to which the generator is coupled or belt driven from it if high speed is desired for it. The installation is placed in an open bay, the lower flange of the draft chest resting upon and being secured to a supporting frame of steel members, the shaft bearing on floor stands or bridge trees. The gate devices of both turbines are operated by a union gate shaft finally connected to the turbine governor. The economical limit of this installation is found from a comparison of the cost of the turbine bay construction and that of turbine casings and penstocks supplying the water to them. The efficiency obtainable from this arrangement, as compared with that of horizontal turbines cased and penstock fed, should be higher, because of the loss of head involved in supplying water through pipes.

The power-house design adapted to this turbine installation may be as shown in Figs. 85 and 86 of Article 76, with such variations in details as will be suggested by the local conditions and the power head.

Diagram 35 gives the approximate exterior dimensions of this installation for turbines of various sizes, from which the required bays can be planned.

Example.—For a pair of 35″ turbines drowned, the height from the supporting frame to the centre shaft line, E in the diagram, is 3′ 3″; the height from the supporting frame to the top of the draft chest, B in the diagram, is 5′ 8″; the diameter of the draft tube collar, D in the diagram, is 8′ 5″; the total length of the installation, A in the diagram, is 15′ 5″; the required width of the turbine bay is 2 B or 11′ 4″; the required length of the turbine bay is A + 5′ or 20′ 5″.

Fig. 118 shows the installation of *three horizontal turbines drowned.*

This is the same arrangement as has just been described, with the addition of the third wheel, which discharges by separate draft tube. The draft chest of the single turbine is of the quarter-turn shape, the union shaft penetrating it by a stuffing-box. This arrangement is as efficient as that of one pair of turbines and frequently preferable in order to secure a higher generator speed.

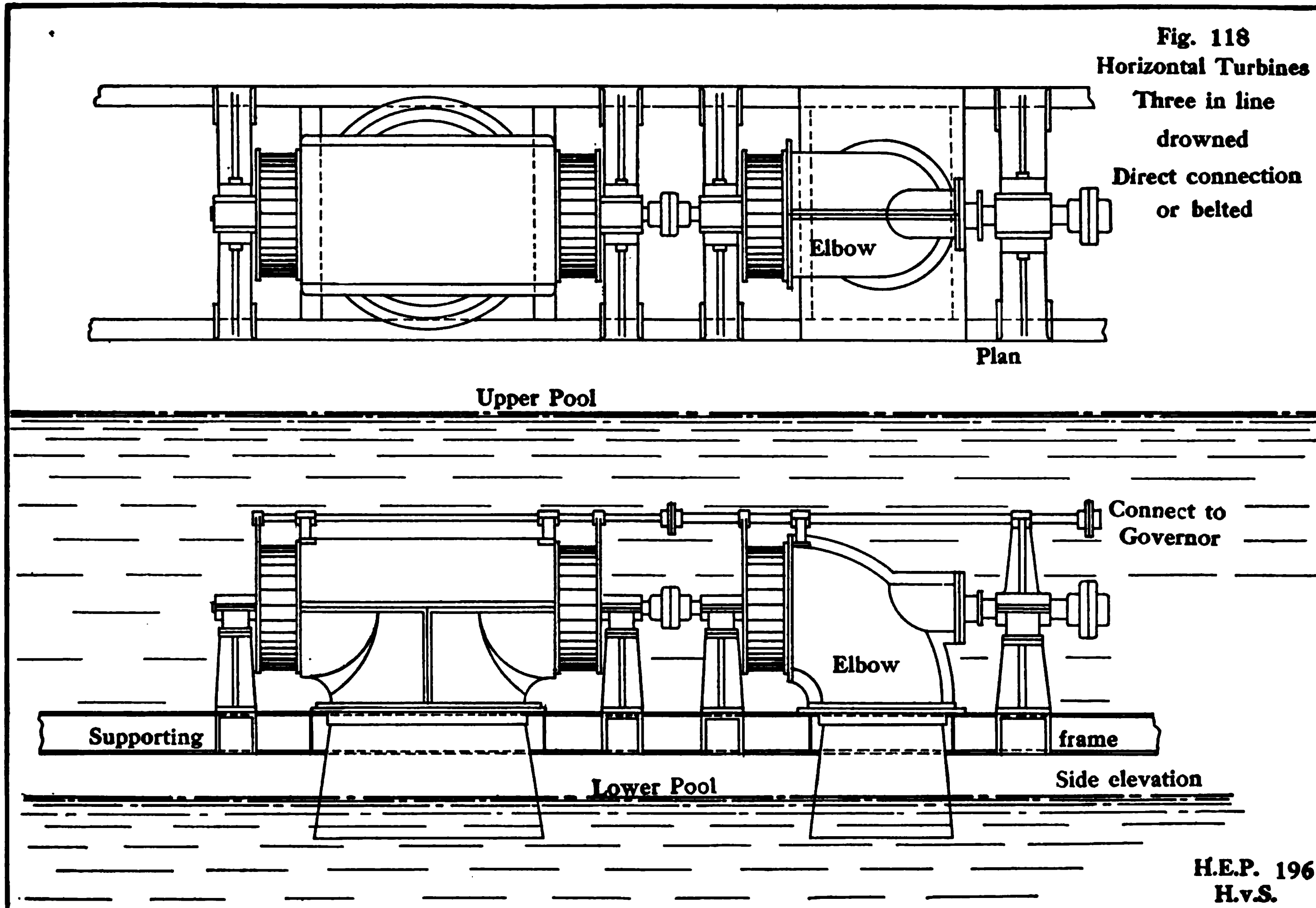


Fig. 118
Horizontal Turbines
Three in line
drowned
Direct connection
or belted

The dimensions for this installation are found from Diagram 35 for the pair and Diagram 36 for a single horizontal turbine drowned.

Example.—For three 30″ turbines, the height from turbine supporting frame to the shaft and to the top of the draft chest is the same as that given for the double turbine on Diagram 35; the diameter of the draft tube collar for the single turbine, D in Diagram 36, is 4′; the total length is the sum of A on Diagram 35 and B on Diagram 36, or 13′ 7″ + 11′ 11″ = 25′ 6″; the width of the required bay is the same as that for the double turbine; the length of the turbine bay is the length of the installation + 5′, or 30′ 6″.

The installation may be of a single horizontal turbine drowned, as shown on Diagram 36. In this case the open bay need only be long enough to allow the water to enter the guide wheel as is indicated by the partial bulkhead in the figure on Diagram 36. The dimensions may be readily found from Diagram 36.

Fig. 119 illustrates the installation of *two pair of horizontal turbines drowned.*

The characteristics of this installation are the same as those of one pair or of three horizontal turbines drowned, the difference being one of dimensions only, and they change merely as to the length, which is double that given on Diagram 35.

In this manner any number of turbines can be united into one power unit. The plant on the Spring River, Kansas, recently constructed, consists of units containing four pairs of double horizontals drowned. The wisdom of such a long line of turbines operating one shaft may be questioned, on account of the many shaft bearings which must be free to line to avoid serious friction losses.

Figs. 120 and 121 show the installation of a *pair of horizontal turbines cased.*

As with vertical turbines, the installation of horizontals may be in cases, single or double, supplied through a penstock entering the case at the top or at the end, the discharge being by one or two draft tubes. This arrangement meets the requirements of heads which exceed the utilization of the drowned type and is available until the pressure head exceeds the economical limit of turbine-case strength. The power-house design for this installation is shown in Fig. 89, of Article 76.

The dimensions for this programme are given in Diagram 37.

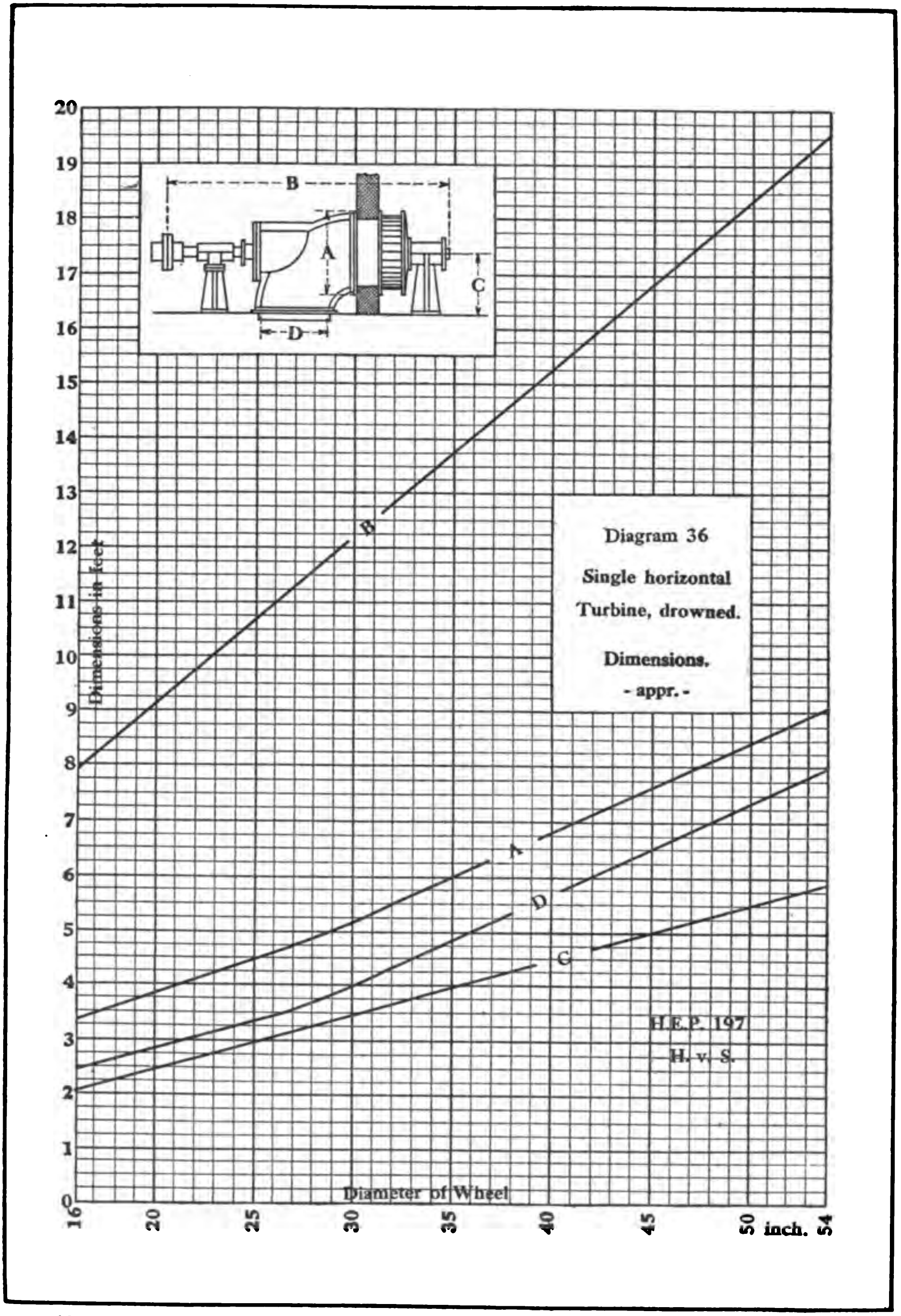


Diagram 36
Single horizontal
Turbine, drowned.
Dimensions.
- appr. -
H.E.P. 197
H. v. S.
Dimensions in feet
Diameter of Wheel
inch.
B
A
C
D
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
16
20
25
30
35
40
45
50
54

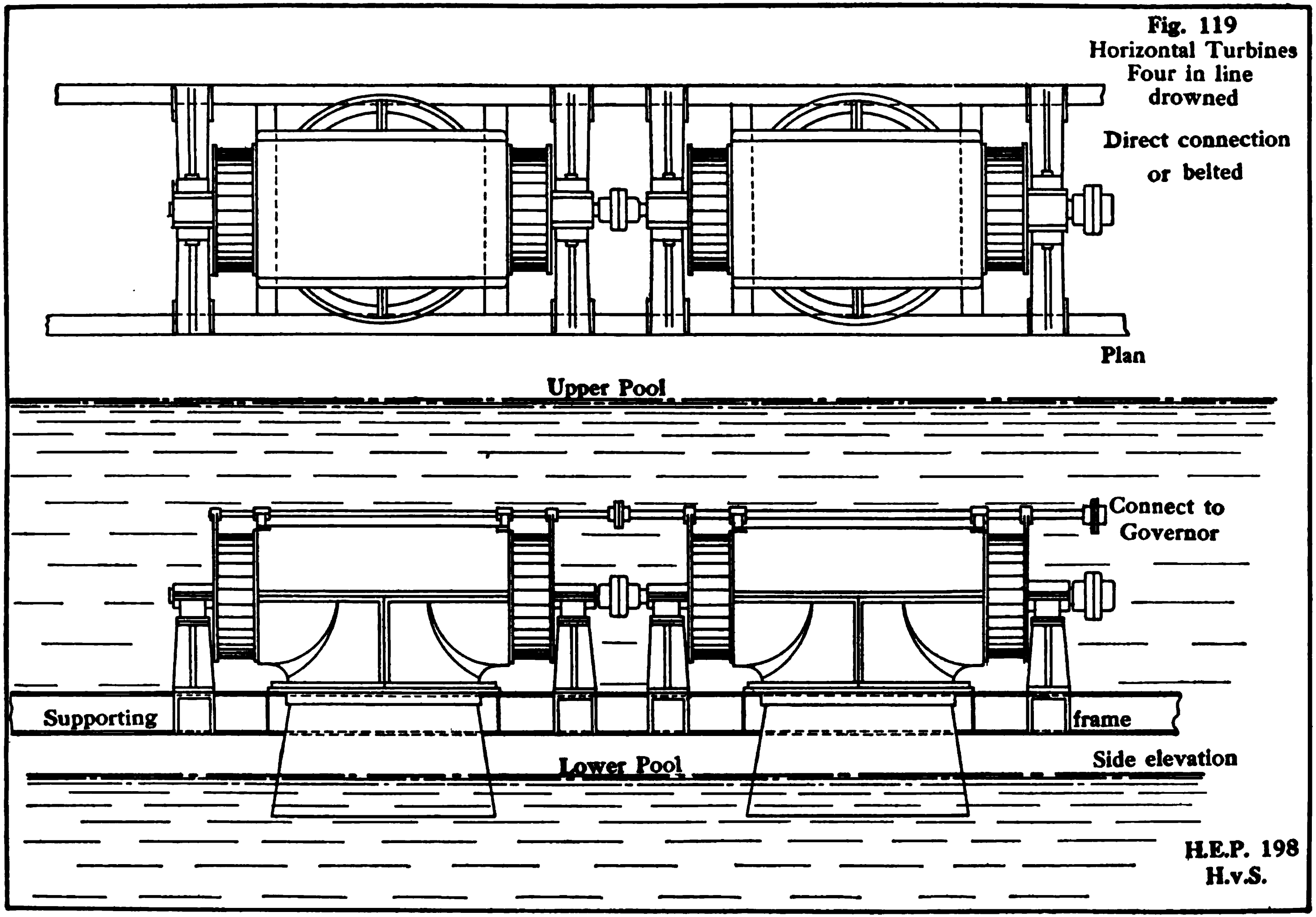


Fig. 119
Horizontal Turbines
Four in line
drowned

Direct connection
or belted

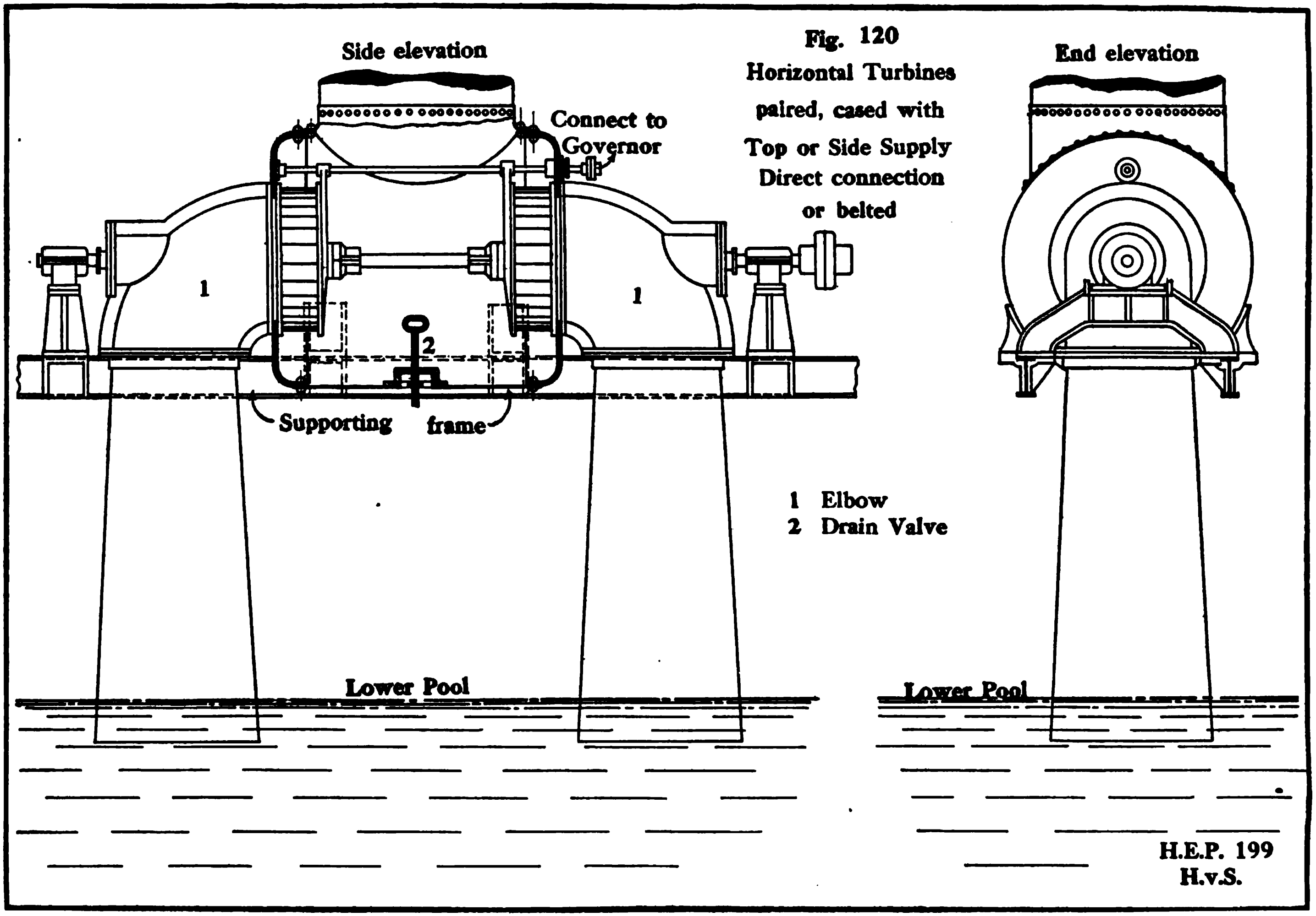


Fig. 120
Horizontal Turbines
paired, cased with
Top or Side Supply
Direct connection
or belted

Example.—For two 40″ turbines cased, the height from the supporting frame to the centre shaft line, G in the diagram, is 4′ 2″; the diameter of the case, A in the diagram, is 10′ 11″; the total length, F in the diagram, is 31′ 8″.

ARTICLE 87. *Reaction Turbine Output.*—The discussions of the theory of turbines in Article 79 and of turbine efficiency in Article 85 have paved the way for the presentation of this final turbine topic, "the output," a thorough understanding of which is highly essential to determine what is the most resourceful hydraulic equipment for a given case.

The turbine output here referred to is *the power yield at highest shaft speed with the least quantity of water at three-quarter gate opening*, which is the definition of the maximum efficiency output. The conditions of this output are rigid as to speed, which, in hydro-electric practice, represents the first essential, because the efficiency of the generator is largely based upon it; in other words, the power yield rises or falls as more or less water is passed through the turbine, while the speed remains approximately constant.

The three-quarter gate discharge basis is the most practical, lying midway between full and half, the latter being the low limit of resourceful efficiency, while it provides a reserve output, from three-quarter gate to full, which is especially valuable in meeting the frequent increase of the generator load. The discharge from three-quarter to full gate is practically proportionate to the gate opening, while that at half gate is somewhat in excess of the half full gate discharge.

The efficiency of this output is approximately 80 per cent., provided the conditions of design and construction on which it is based are met.

The power constants here given are those of *reaction turbines*, and the values are those for *one foot head;* they are distinguished from the output of the same turbine for more than one foot head by the prime mark, thus P′, Q′, S′, representing the power, discharge, and speed output of the turbine under one foot head, while P, Q, and S represent the output of the same turbine for a greater head than one foot.

Power, in mechanical horse-power, for unit head, P′, from

$$P' = \frac{Q' \times 62.5}{550} \times \text{efficiency} = Q' \times 0.1136 \times 0.80 = 0.09\,Q'.$$

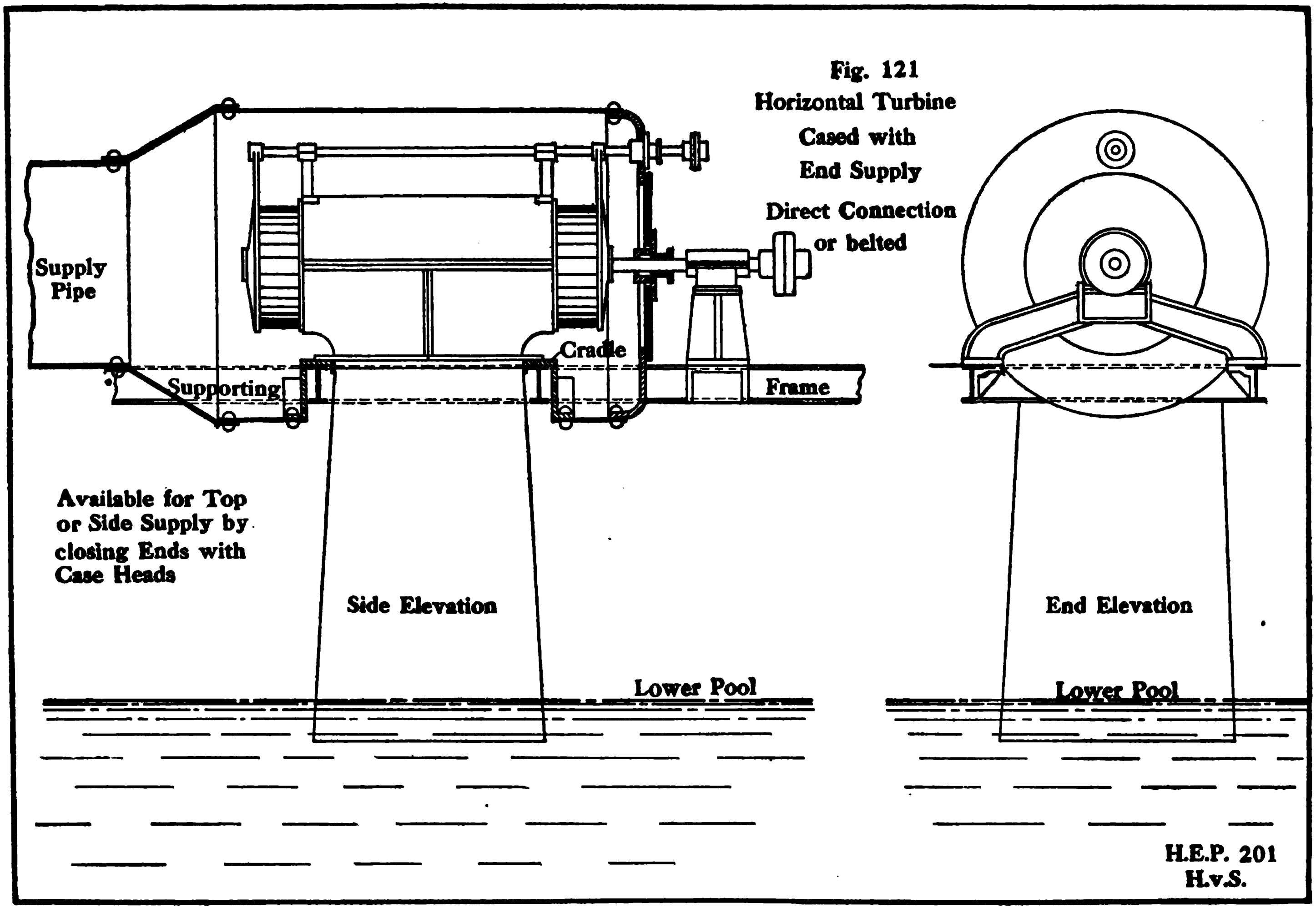


Fig. 121
Horizontal Turbine
Cased with
End Supply
Direct Connection
or belted

For any head greater than one foot the discharge of the same turbine increases with $\sqrt{H}$, and the power output for any head H therefore is

$$P = P' \times \sqrt{H^3} = 0.09\,Q' \times \sqrt{H^3}.$$

Discharge, in cubic second feet, from power constant

$$Q' = \frac{P'}{0.09} = 11.1\,P'.$$

For any head H, $Q = Q' \times \sqrt{H} = 11.1\,P' \times \sqrt{H}$.

Speed, in revolutions per minute, S′, for unit head, from the peripheral speed due to $0.89\sqrt{2g}$ = 7.1378 second feet,
or = 428.268 minute feet,
and, as turbine diameters are expressed in inches, = 5,139 minute inches,

$$S' = \frac{5139}{3.1416} = \frac{1635}{\text{diam. in inches}}.$$

Diameter is expressed, as will be seen under Design Constants, as

$$D = 6.36\sqrt{Q'} \text{ and inserting in above } S' = \frac{257}{\sqrt{Q'}}.$$

For any head H, $S = S' \times \sqrt{H} = 257\sqrt{H} \div \sqrt{Q'}$.

Diagram 38 gives the standard maximum efficiency output of American reaction turbines, which are designed and constructed on the general lines detailed further on.

Example from Diagram 38.—*For a* 35-*inch Turbine:*

Power constant is	2.8	mechanical horse-power,
Speed constant is	47	revolutions per minute,
Discharge constant is	31	cubic second feet.

The output of the same (35″) turbine for a 16-feet head is:

Power, constant,	$\times \sqrt{16^3}$ = 179.2	m.h.p.,
Speed, constant,	$\times \sqrt{16}$ = 188	r.p.m.,
Discharge, constant,	$\times \sqrt{16}$ = 124	c.s.f.

For a 60-*inch Turbine:*

Power constant is	8.28	m.h.p.,
Speed constant is	27	r.p.m.,
Discharge constant is	92	c.s.f.

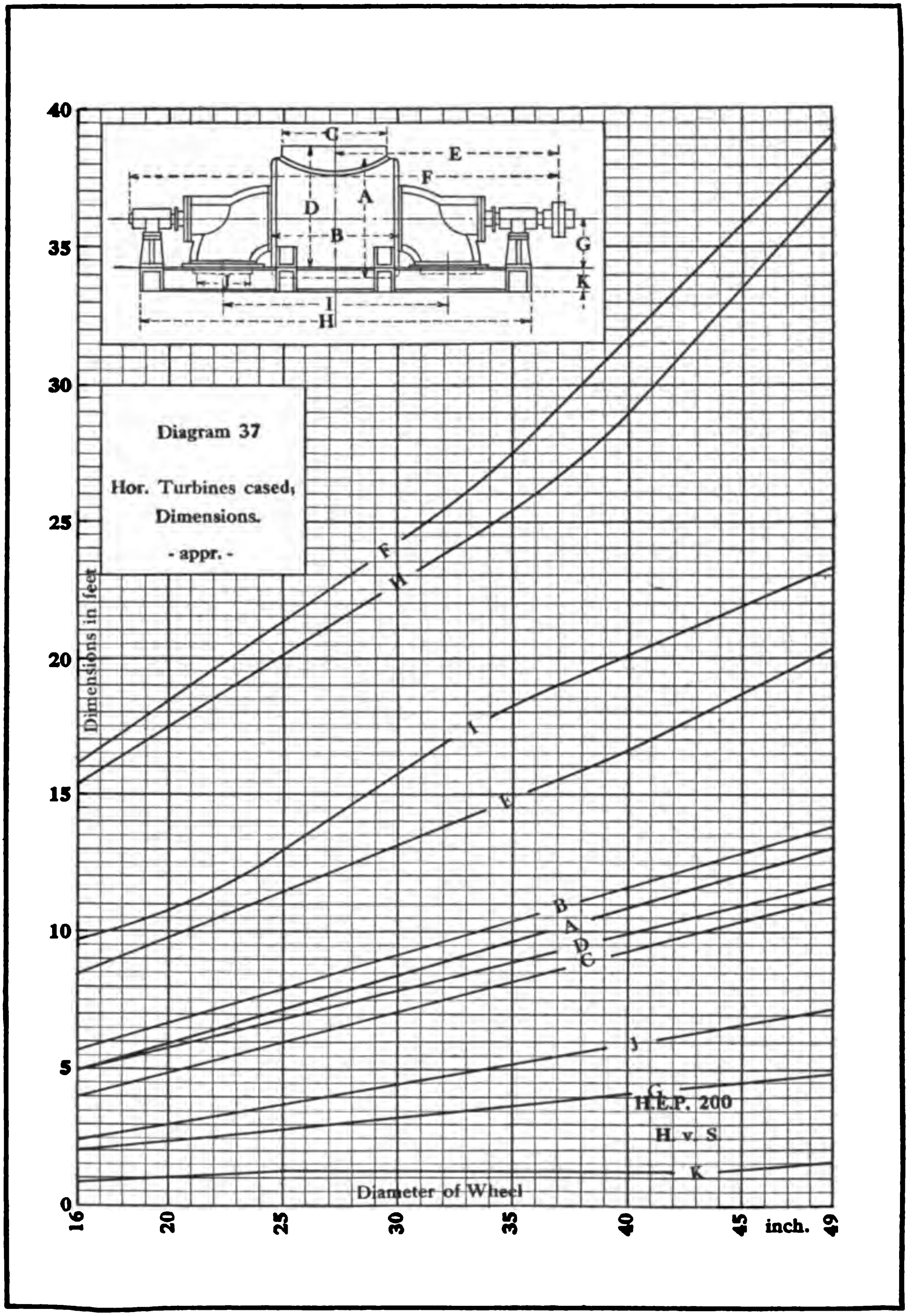


Diagram 37
Hor. Turbines cased,
Dimensions.
- appr. -
Dimensions in feet
Diameter of Wheel
inch.
H.E.P. 200
H. v. S.
F
H
I
E
B
A
D
C
J
G
K
0
5
10
15
20
25
30
35
40
16
20
25
30
35
40
45
49

The output of the same (60″) turbine with a head of 25 feet is:

Power, constant	$\times \sqrt{25^3}$	=	1035 m.h.p.,
Speed, constant	$\times \sqrt{25}$	=	135 r.p.m.,
Discharge, constant	$\times \sqrt{25}$	=	460 c.s.f.

The efficiency of this standard output is (for one-foot head)

from 62.5 Q′ ÷ 550, efficiency, E =	8.8 P′		÷ Q′ (second feet)	
or	= 528 P′		÷ Q′ (minute feet)	
Applied to above examples, 35″ E =	8.8 ×	2.8	÷ 31	= 79.32
Applied to above examples, 60″ E =	8.8 ×	8.28	÷ 92	= 79.20
For the 35″ turbine with 16′ H, E =	8.8 ×	179.2	÷ 124 × 16	= 79.48
For the 60″ turbine with 25′ H, E =	8.8 ×	1935.	÷ 460 × 25	= 79.19

This is representative of the degree of accuracy of deductions taken from Diagram 38, the standard efficiency being 80.

ARTICLE 88. *Reaction Turbine Design.*—The elements of the design of the reaction turbine yielding the standard maximum efficiency output are:

The diameter in inches, from the area required to pass the volume

of water with one-foot head, $Q' \div \sqrt{2g} = 0.1247\,Q'$ (c. s. f.),
expressed in cubic second inches $= 17.9568\,Q'$ (c. s. f.).

The circular area which would pass this quantity (theoretically)

$$\text{is } D^2 \times 0.7854 \qquad \text{and } D^2 = 17.9568\,Q' \div 0.7854 = 22.863\,Q'$$

or D (theoretical diameter of turbine in inches) = 4.78 Q′.

To compensate for the obstructions to the free passage of the water, the above theoretical value is increased by the coefficient of 1.33 and the turbine diameter = $1.33 \times 4.78 \sqrt{Q'}$, or $D = 6.36 \sqrt{Q'}$.

The vent area is found from the theoretical velocity $\sqrt{2g}$ and the coefficient 0.60, or for unit head and in minute feet velocity, $V = 60 \times 0.60\sqrt{2g} = 288.7$ minute feet.

The discharge through an opening of one square inch area is

$288.7 \div 144 = 2$ (appr.), and the required vent area
in square inches $= 60\,Q' \div 2 = 30\,Q'$.

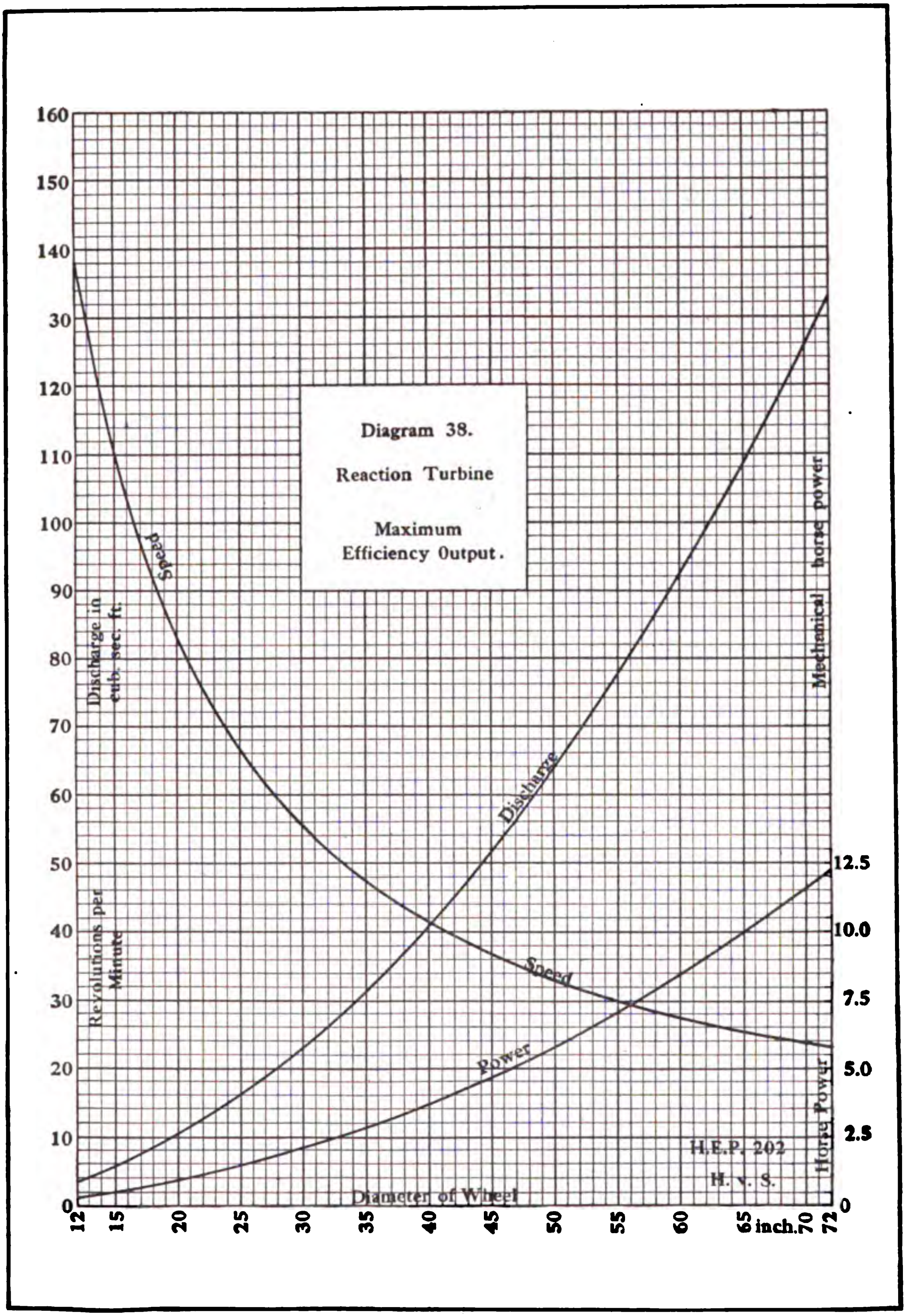


Diagram 38.
Reaction Turbine
Maximum
Efficiency Output.
Discharge in
cub. sec. ft.
Revolutions per
Minute
Mechanical horse power
Horse Power
Speed
Discharge
Power
Diameter of Wheel
H.E.P. 202
H. v. S.
inch.
160
150
140
30
120
110
100
90
80
70
60
50
40
30
20
10
0
12.5
10.0
7.5
5.0
2.5
0
12
15
20
25
30
35
40
45
50
55
60
65
70
72

Guide-wheel or gate openings represent the total vent area, which is generally divided between 12 such openings, the area of each is therefore = 2.5 Q′ (square inches).

The dimensions of the guide-wheel or gate openings are generally one of width to four of height.

The runner buckets should be fewer in number than the guide-wheel or gate openings, so that anything which passes through the latter, like chips, ice, etc., is likely to pass through the turbine proper without becoming clogged in it.

The entrance angle to the runner buckets should be such as to avoid shock of the entering water.

The clearance between the guide wheel and the runner should not exceed one-eighth of an inch.

The bucket vanes are of a helical form, changing the direction of the flowing water nearly 180°; their axial depth is fixed by the axial length of the guide openings, for the axially straight part, and the least additional length which is necessary to secure a true helical curve.

The draft-tube diameter at the entrance should be such that the velocity does not exceed five feet; the exit diameter should be 1.5 of the entrance diameter; the sum of the entrance diameter and the length above tail-water should be approximately

29 feet for diameters up to two feet,
27 feet for diameters from two to three feet,
25 feet for diameters from three to four feet,
23 feet for diameters from four to six feet,
22 feet for diameters from six to twelve feet.

The structural finish of the turbine parts should be as defined in Article 93.

ARTICLE 89. *Output of Tangential Impulse Turbines.*—The efficiency of this type of turbines is somewhat higher than that of reaction turbines, owing to the fact that the water does not pass through guides and buckets, but is jetted directly upon the bucket faces; the hydraulic losses are therefore much minimized, while those of unused energy and of mechanical origin are about the same as for the reaction turbines. Draft tubes are not of the same value with these turbines as they are with the reaction types, unless the turbine runner is housed in an air-tight casing,

and then the loss of head is that represented by the height between the elevation of the nozzle delivering the jet of water and the elevation of the tail water.

The constants of the maximum efficiency output of tangential turbines are based upon the standard efficiency of 85. The deductions of these constants follow the same lines as detailed in Article 87.

Power, in mechanical horse-power, for unit head, $P' = 0.0966\,Q'$.
Speed, in revolutions per minute, for unit head, $S' = 10.6 \div \sqrt{Q'}$.
Discharge, in cubic second feet, for unit head, $Q' = 10.35\,P'$.

Diagram 39 gives these constants for unit head for the standard size tangential turbines as manufactured in the United States.

Example from Diagram 30.—For a 24-inch Tangential Turbine:

Power constant is	0.0078	mechanical horse-power,
Speed constant is	38	revolutions per minute,
Discharge constant is	0.08	cubic second feet or
	4.8	cubic minute feet.

The output of this same 24-inch tang. turbine with a head of 900 feet is:

Power, constant	$\times \sqrt{900^3} =$	21.06	m.h.p.,
Speed, constant	$\times \sqrt{900} =$	1140	r.p.m.,
Discharge, constant	$\times \sqrt{900} =$	2.4	c.s.f.

For a 60-inch Tangential Turbine:

Power constant is	0.047	m.h.p.,
Speed constant is	15.3	r.p.m.,
Discharge constant is	0.49	c.s.f.

The output of this 60-inch tangential turbine with a head of 2500 feet:

Power, constant	$\times \sqrt{2500} =$	5875	m.h.p.,
Speed, constant	$\times \sqrt{2500} =$	765	r.p.m.,
Discharge, constant	$\times \sqrt{2500} =$	24.5	c.s.f.

The chief elements of the design of tangential turbines to yield the output expressed by the foregoing constants are for

the diameter, in inches, $D = 86.5\,Q'$ (Q in cubic second feet),
the nozzle, in inches, $N = D \div 18$.

Several nozzles may carry water to one wheel, as many as five being frequently used, whereby the same diameter tangential turbine dis-

charges about eight times the normal volume, the power output being increased correspondingly while the speed is slightly lower.

ARTICLE 90. *Summary of Turbine Output Constants.* — These constants represent the standard maximum efficiency output of American reaction and impulse turbines at efficiencies of 80 and 85 respectively; the expressions are for unit head, or a fall of one foot, being

power, in mechanical horse-power, with one foot head, P',
speed, in revolutions per minute, with one foot head S',
discharge, in cubic second feet, with one foot head, Q'.

Reaction Turbines.		Tangential Impulse Turbines.
$0.09\ Q'$	Power constant	$0.0966\ Q'$
$257 \div \sqrt{Q'}$	Speed constant	$10.6 \div \sqrt{Q'}$
$11.1\ P'$	Discharge constant	$10.35\ P'$
$6.36 \sqrt{Q'}$	Diameter	$86.5 \sqrt{Q'}$
	Nozzle (single)	$4.75 \sqrt{Q'}$

For any head H the corresponding output is obtained for

Power, from power constant $\times \sqrt{H^3}$ for reaction and impulse turbines.
Speed, from speed constant $\times \sqrt{H}$ for reaction and impulse turbines.
Discharge, discharge constant $\times \sqrt{H}$ for reaction and impulse turbines.

Diameter and nozzles remaining as per constants.

ARTICLE 91. *Determining the Turbine Equipment.* — As has been stated before, the chief criterion in selecting the turbine equipment for a hydro-electric plant is the speed. From the constants in Article 90 it will be noted that the ratio of speed of reaction and tangential turbines is about as 24 to one, and therefore the first will prove generally preferable for hydro-electric installations until the head, and therefore the speed, becomes too high both for the reasonable wear of the turbine and the speed standard of the best adapted electrical apparatus for the conditions. No demarcation line can, however, be drawn between the adaptability of the two types of turbines excepting the general limitation of the high head, as for heads above 300 feet reaction turbines will not prove preferable over the tangential; however, there may be conditions, with heads much lower than that stated, where the tangential type should be preferred, as it must not be overlooked that from it can be secured an increase of not less than five per cent. output in power, which may, in small plants, form the deciding argument.

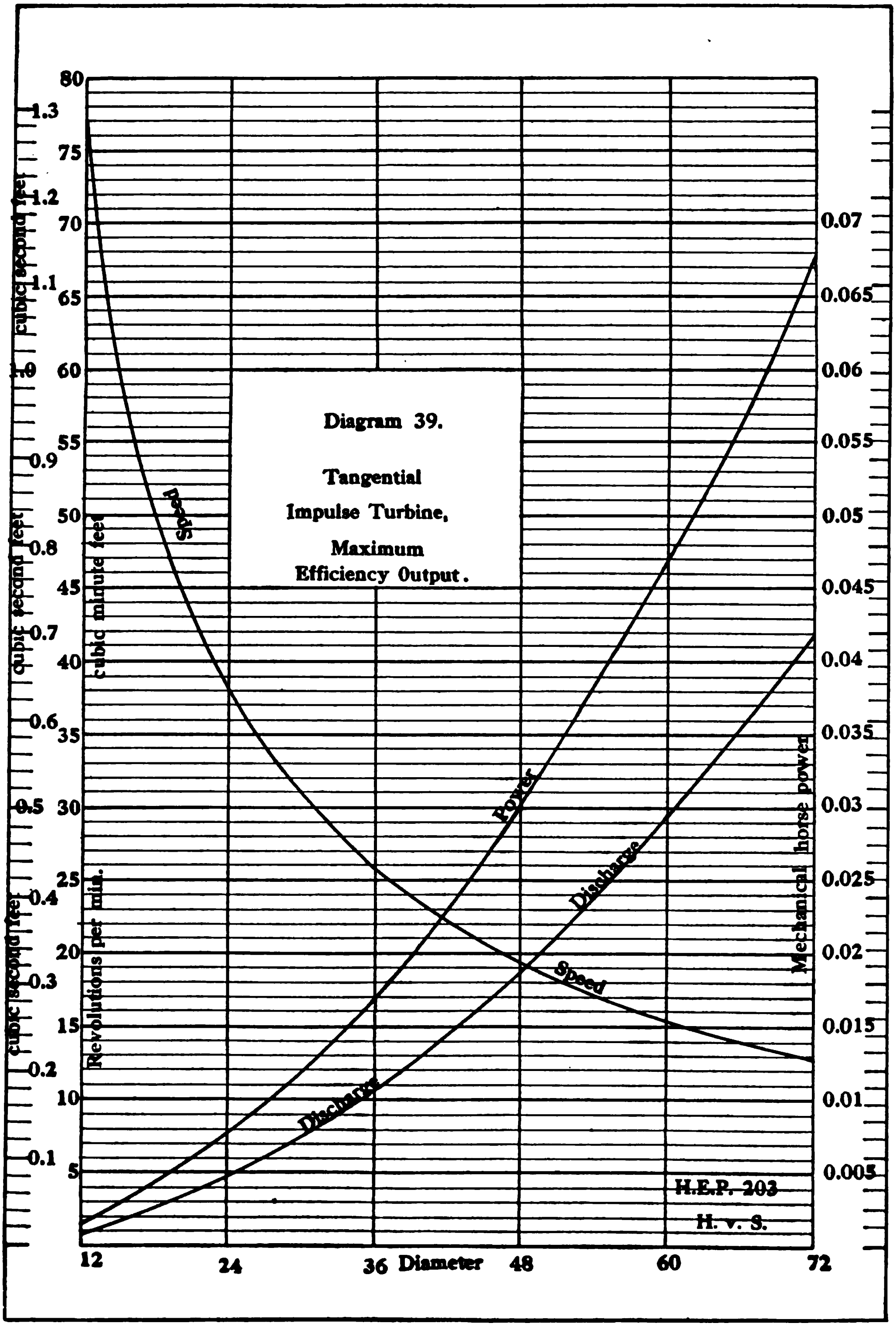


Diagram 39.
Tangential
Impulse Turbine,
Maximum
Efficiency Output.
Speed
Power
Discharge
H.E.P. 203
H. v. S.
Diameter
12
24
36
48
60
72
Revolutions per min.
cubic second feet
cubic minute feet
Mechanical horse power
80
75
70
65
60
55
50
45
40
35
30
25
20
15
10
5
1.3
1.2
1.1
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.07
0.065
0.06
0.055
0.05
0.045
0.04
0.035
0.03
0.025
0.02
0.015
0.01
0.005

The power functions, available head and flow, being known, the constant output discharge volume is found by dividing available flow by $\sqrt{H}$; this is the value of Q', and the investigation is ready for consultation of Diagram 38 or 39, as the head may be low or medium or high.

Example 1.—*Available flow is* 460 *c.s.f. and head is* 16 *ft.*

$$Q' = 460 \div \sqrt{16} = 115.$$

If this quantity is outside of the diagram scope, it exceeds the discharge output of a commercially standard reaction turbine, and it must therefore be reduced by division into the least number of parts required to bring the unit quantity within the limit of the diagram.

Returning to Example 1, the unit discharge of 115 c. s. f. is found on the left-hand index and traced to the right until the discharge curve is intersected, thence leading downward to the lower index where the turbine size is found of which this quantity is the discharge constant, and from this to the intersections of the dimension index with the power and speed curves these constants are obtained, and from them the output for the available head of 16 feet. In this case a 67-inch turbine represents this discharge, and its constants are:

$$\begin{array}{lll} P' = 10.5 & S' = 24.5 & Q' = 115, \\ \text{and the 16-ft. output, } P = 672 & S = 98 & Q = 460; \end{array}$$

the speed is therefore 98 revolutions, which is too low for direct connected electric apparatus.

Taking half of the unit discharge, 58 c. s. f., Diagram 38 gives a 48-inch turbine which meets it; the output constants are:

$$\begin{array}{lll} P' = 5.25 & S' = 36 & Q' = 58, \\ \text{the 16-ft. output } P = 336 & S = 144 & Q = 232. \end{array}$$

If this speed of 144 r. p. m. is still too low, one-fourth of the flow constant is taken or 29 c. s. f., for this
Diagram 38 gives a 34-inch turbine; the output constants are:

$$\begin{array}{lll} P' = 2.6 & S' = 49 \text{ and } & Q' = 29, \\ \text{the 16-ft. output is } P = 166.4 & S = 196 & Q = 116. \end{array}$$

Two 34″ turbines in one unit yield 332.8 m. h. p.

The ratio of turbine diameters as above, 67, 48, and 34, is the constant representing the ratio of the diameter of one large turbine to the diameter of two smaller turbines with like power output, and vice versa, which is expressed approximately by

$$D = 1.42\,d \quad \text{and} \quad d = 0.71\,D.$$

Example.—The power yield of one 54-inch turbine is equalled by that of two 38-inch turbines from 54 × 0.71, and the power yield of two 23-inch turbines is equalled by that of one 33-inch turbine, from 23 × 1.42.

The final comparative factors are speed and output expressed in terms applied to the commercial standard types of generators, that is kilowatt, in accordance with the maximum efficiency output of turbines, which is higher than the water-power-electric power ratio given in Diagram 3 of the first part of this volume, where a lower turbine efficiency was adopted in order to secure safe conservative estimates for the preliminary investigation of a hydro-electric opportunity.

The following arrangement may be found convenient for the equipment determination.

Available Unit.	Q 1200 / 200	H 36 / 1	D	P	S	K.W. Per unit.	K.W. Total.
One turbine	...	..	90″				
Two turbines @.....	600	36	64″	1,944	153	1,376	2,752
Three turbines @....	400	36	52″	1,296	188	917	2,751
Four turbines @	300	36	45″	972	218	688	2,752
Five turbines @.....	240	36	40″	778	245	550	2,750
Six turbines @......	200	36	37″	648	265	459	2,754
Seven turbines @ ...	171	36	34″	561	294	397	2,779
Eight turbines @....	150	36	31″	475	324	346	2,768
Nine turbines @.....	133	36	29″	432	336	305	2,752
Ten turbines @	120	36	28″	388	360	275	2,750
Eleven turbines @ ..	109	36	27″	356	372	252	2,772
Twelve turbines @ ..	100	36	25″	324	396	229	2,748

From this analysis of turbine output all feasible unit combinations can be drawn:

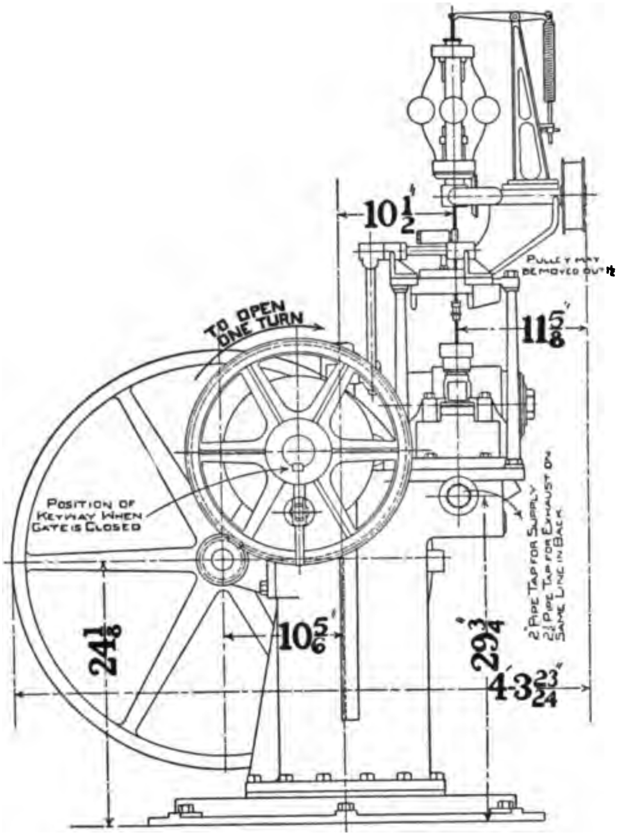
		Speed.	Unit output.
Twelve units of single....................................	25″	396	229
Six units of double.......................................	25″	396	458
Five units of double......................................	28″	360	550
Four units of double......................................	31″	324	692
Four units of three.......................................	25″	396	687
Three units of four.......................................	25″	396	916
Three units of three......................................	29″	336	905
Two units of three..	37″	265	1,377
Two units of two	45″	218	1,376
Two units of single.......................................	64″	153	1,376

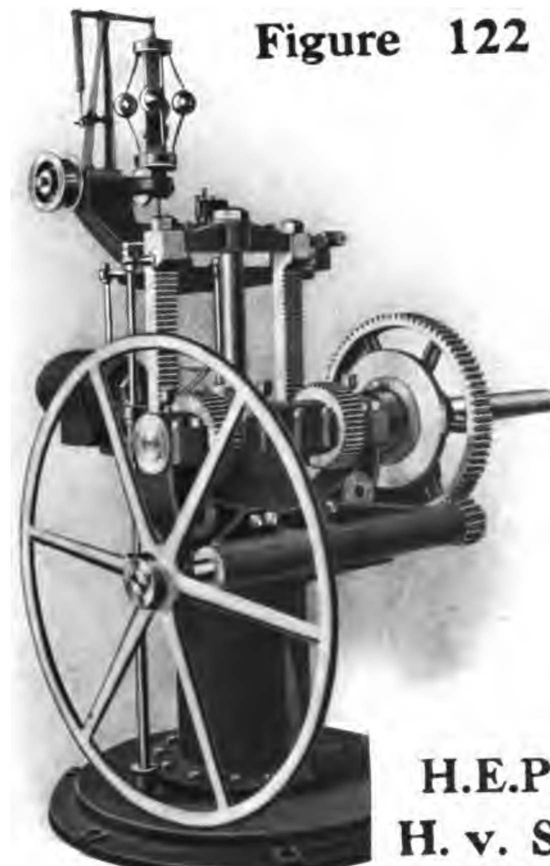
This represents all the practicable turbine combinations for this set of conditions, covering a range of speed from 153 to 396 and of generators from 229 to 1376 K.W. capacity, and it exhausts the investigation at the turbine end of the equipment, and is continued by the further examination of standard generator units and the final selection of such an arrangement as best harmonizes the three factors,—market requirements, turbine, and generator adaptability.

With high heads, of 300 feet and more, the same process is extended to tangential impulse turbines, in which Diagram 39 may be utilized; here the scope of possible combinations is somewhat narrower, unless the feasibility of supplying the water by more than one jet is taken into consideration, which leads into the volute or central discharge turbine, types more frequently employed abroad, but which may now also be procured in this country, and represents the high efficiency of the tangential turbines coupled with a large discharge capacity.

ARTICLE 92.—*Turbine governors* are required in connection with the operating of the turbine equipment of a hydro-electric plant when the head or the work to be done by the generated power, the load, fluctuates; one or both of these conditions prevail in probably every hydro-electric power plant. Like other governors, turbine governors are expected to regulate the volume of the power-generating substance, in this case the water, which regulation, in reaction turbines, must be applied at the turbine gates, controlling the gate openings, while in impulse turbines of the tangential type, where the water is supplied to the buckets by the way of a nozzle, the regulation by the governor must be of the nozzle's opening, which is generally secured by the axial movement of a plug or needle passing in the interior of the nozzle back or forth as actuated by the governor's movements. These conditions are widely different from those prevailing in the regulation of steam-engines, steam being elastic and compressible, while water is not; the steam valves to be operated by the governor being accurately fitting machinery parts easily moved and controlled, while the gates of turbines are large and heavy, and certainly not to be compared in fit and finish of motion to the valve; a turbine gate is exposed to the water pressure and when once set in motion is not readily stopped or controlled. All these conditions make it necessary that a turbine governor be supplied with some other source of energy, to be applied to the regulating of turbine gates, than that represented by the usual centrifugal balls of the steam governor, and all turbine

Figure 122





Lombard Governor

H.E.P.
H. v. S.
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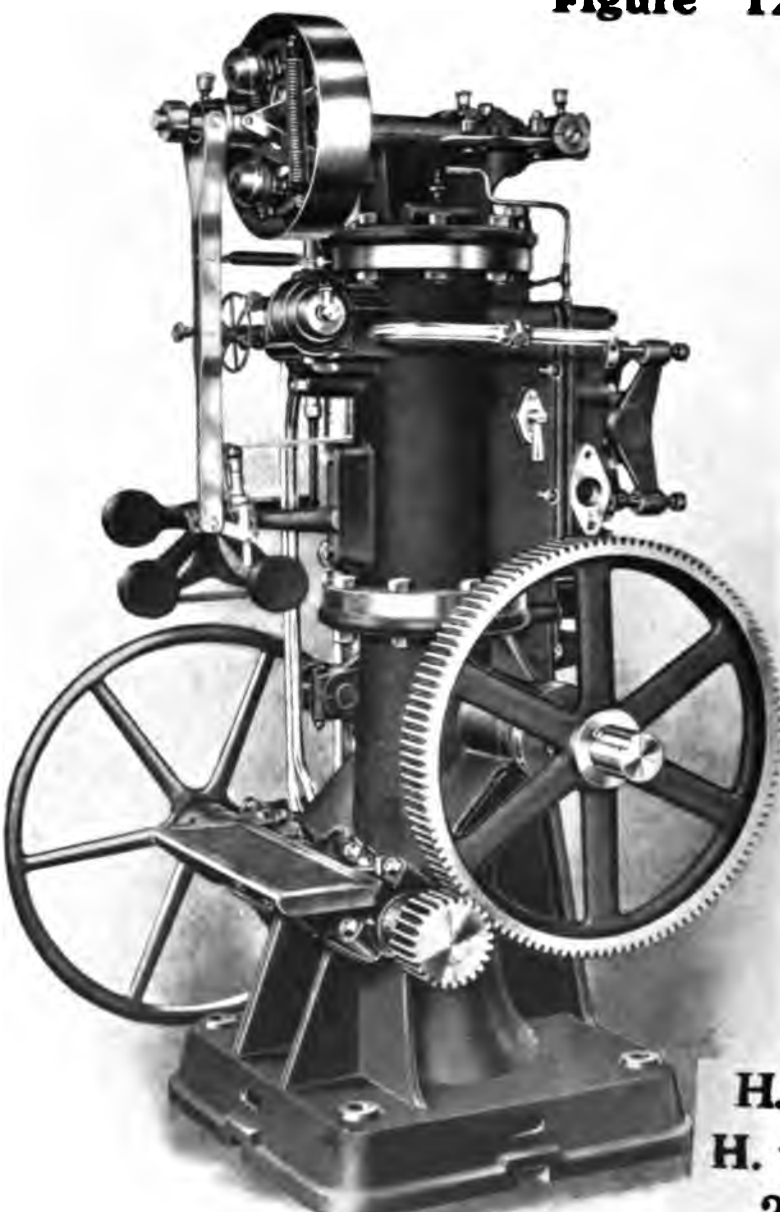
Figure 123



H.E.P.
H. v. S.
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Sturgess Governor

governors have such additional power source made available through the initiative of the centrifugal speed regulation. This is the *relay* energy source of the governor, and may be of mechanical, hydraulic, electric, or pneumatic origin, the first two being those applied in American practice.

The manufacture of governors of this class is restricted in this country to a few concerns, and it will serve the purpose best to describe, very briefly, the pertinent characteristics of the different types, which here follow; the illustrations of these governors have been furnished by the respective makers of the machines.

The Lombard governor is manufactured by the Lombard Governor Company, at Ashland, Mass.; it is of the hydraulic type.

Fig. 122 shows the elevation of one of the several types and a section with the dimensions of the important features.

This governor consists of the following parts: A centrifugal speed regulator, a regulating valve with adjustable valve stem, a pressure tank and receiver, a hydraulic cylinder, power pump, antiracing mechanism, and terminal connections of racks, pinions, clutch, and hand-wheel shaft.

The governor's operation is briefly described thus: The centrifugal head is connected to a regulating valve by a valve stem in two parts connected by a screw coupling, which is adjustable and whereby the normal governor speed may be fixed or altered. The fluctuation of the centrifugal head actuates the regulating valve, which permits fluid under pressure to pass from a pressure tank to the hydraulic cylinder which controls the gate operations. The oil is pumped into the pressure tank by a suitable force pump; where high pressure heads are available, water pressure may be utilized; the oil which is exhausted by the hydraulic cylinder passes back into the receiver from whence it is again returned by the pump to the compressor tank; in this manner the relay energy is practically maintained constant.

The Sturgess governor is also of the hydraulic type; it is manufactured by the Ludlow Valve Manufacturing Company, at Troy, N. Y.

Fig. 123 shows this type of governor in elevation. It consists of a centrifugal governor, a pilot valve operated by the centrifugal governor, an operating cylinder controlled by the pilot valve and operating puppet valves of the main cylinder, a system of floating levers between the operating cylinder and the puppet valves, puppet valves which control the pressure in the main cylinder, a main cylinder with piston, rack, and pinion operating the gate shaft, and a compensator to prevent racing.

The above outline of the principal parts practically describes the operating method. The centrifugal governor is belted to the turbine shaft, the pulleys being proportioned to secure the desired speed of the governor; when the governor operates at the normal speed, the pilot valve stands between the two ports of the operating cylinder, while the smallest deviation from normal speed raises or lowers the pilot valve and affords a passage, to the fluid under pressure, to the hydraulic cylinder connected to the turbine gate shaft.

The Woodward governor is manufactured by the Woodward Governor Company, of Rockford, Ill. It is a mechanical governor—that is, the relay energy is of mechanical source.

Fig. 124 shows the elevation of this governor, known as the compensating type. The power which operates the governor is taken from the turbine shaft by a belt to a pulley on the main governor shaft, to which a double bevelled friction wheel is keyed through which the power for the gate operation is applied. The friction wheel is made of compressed paper. On both sides of the friction wheel are friction pans whose faces are of the same bevel as those of the friction wheel; one of these friction pans effects the opening, the other the closing, of the turbine gates. The friction pans are pressed into the hubs of spur pinions which run free on the governor shaft and engage with gears of a back shaft, one direct and the other through an intermediate gear. Friction wheels and pans are cleared by adjusting rollers. The governor's regulating action is initiated by the centrifugal balls, their deviation from normal speed forcing the friction wheel against the respective friction pan and this in turn acting upon the gate operating shaft. The governor is self-contained, consisting of the following principal parts: a revolving double feed cam, a vertical rock shaft, a main friction shaft, a speed governor, a compensating device, a friction wheel, and the friction pans, with gears, pinions, and connections.

The Lombard-Replogle governor is manufactured at the Replogle Governor Works, at Akron, Ohio; it is a mechanical governor.

Fig. 125 shows the elevation of this governor, in which the speed governor is of the horizontal type, being placed in the main pulley which is driven from the turbine shaft. The action of the speed governor is to press friction wheels into contact at any deviation from the normal speed, when the power developed from these primary or tripping disks brings the secondary or main operating drive into action. The main

Fig. 124.—Woodward Governor.

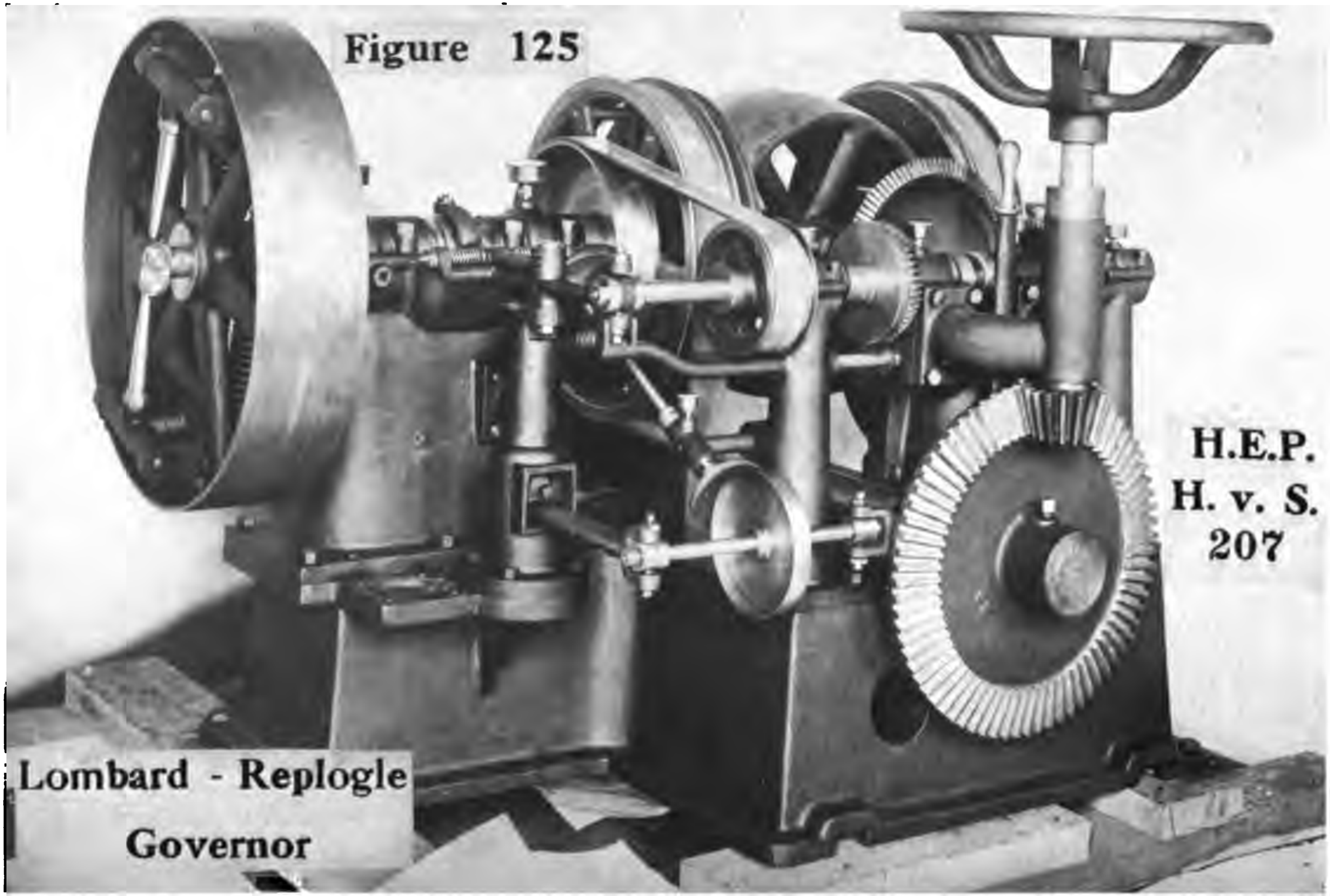


Figure 125

Lombard - Replogle Governor

drive consists of two concave disks which are lined with leather and which rotate in opposite directions and engage a spherical pulley placed concentrically between them. The tripping frictions force this spherical pulley out of the centres of the leather-lined friction disks, causing contact with the moving surfaces and imparting the power which operates the turbine gates. The movement of the spherical pulley in any direction returns it to its normal position at the centre of the friction disks, this being its position of rest. In addition to the effect of the primary disks in forcing the spherical pulley out of the centre of the friction disks, it also neutralizes the speed of the governor balls and, it is claimed, prevents hunting or racing.

This, like any of the other turbine governors, may be regulated, as to the standard speed of the governor element, by electric devices operated from the switchboard of the operating station.

The power required to operate turbine gates depends upon so many different conditions that a practical formula cannot be developed. From extensive experiments made with turbines fitted with balanced swinging or wicket gates, which were carried on under the supervision of the author, it was found that the power required to operate the gates of 33-inch reaction turbines under a head of 16 feet, from their closed to their full open position, was 500 foot pounds; with gates of the same type and correctly swung and balanced this power will vary directly as the head and as the cube of the gate's axial diameter. Cylinder gates require more power, and there are other styles of turbine gates which take still more, though these are becoming obsolete.

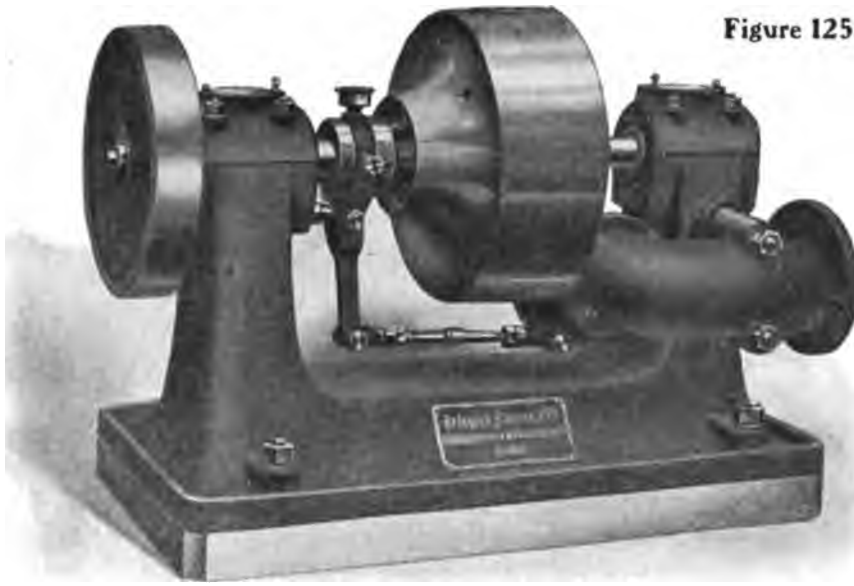
Governing tangential impulse turbines is now generally accomplished by regulating the area of the nozzle through the medium of a plug or needle operating axially in its interior. This may be accomplished by any of the governors described, or by a specially designed hydraulic or electric motor secured directly to the terminal of the supply pipe near the nozzle. Considerations of equally great importance in connection with the governing of these types of turbines relate to the necessary compensation in the supply pipe for the sudden changes and speeds of the volume of the water passing through it and which will be consequential of the governing method, that is the regulation of the volume supplied to the turbine buckets by the way of the jet issuing from the nozzle. It needs no specific discussion to point to the dangerous conditions which may be created for the safety of the supply pipe and its connections.

All of these governors regulate the turbine speed by controlling the volume of water passing into the turbine, but there is another source of energy, the head, and a governor has recently been developed which regulates the turbine speed by controlling the effect due to head. This type is called the *air-governor* and is manufactured by the turbine firm Briegleb, Nansen & Company, of Gotha, Germany.

Description and illustrations below furnished by the makers:—

"The principle of the air-governor consists in admitting air into the draft tube near its junction with the turbine case, by the opening of an

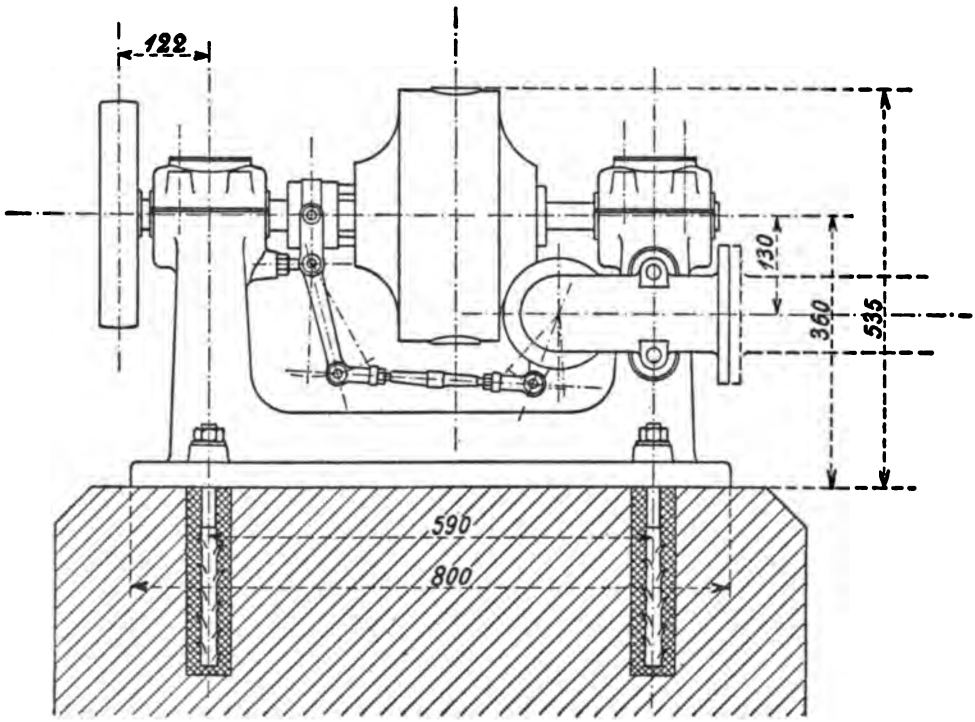
Figure 125a

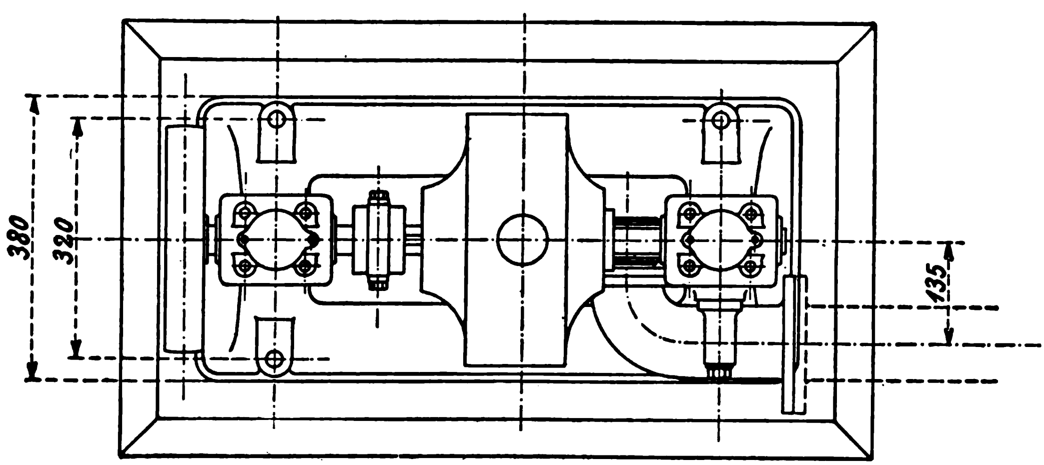


air vent actuated from a balance wheel running on a horizontal shaft and belt-driven from the turbine shaft. Any increase of the normal load speed opens the air vent and thus checks the speed increase by reducing or entirely removing the effect of the head in the draft tube. The advantages of the air-governor are that its operations are entirely automatic, calling for no attendance whatever, that it is not affected by dust, as for instance in saw or lumber mills, it requires only a very small amount of operating power, is self contained and simple, and its first cost is low."

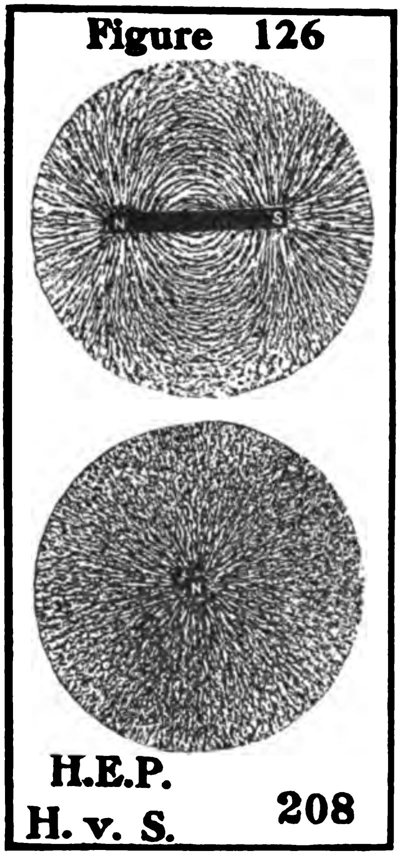
The illustrations give section and base plan with dimensions in centimetres. The weight of the governor here shown which answers practically any turbine installation, is 420 lbs. and the price quoted is $165.00 crated for shipment, f.o.b. cars, the makers' works.

Diagram 40





Hydraulic relief valves may be placed near the nozzle and connected to the supply pipe, or a *by-pass* actuated by a valve responding to an excess pressure; *stand-pipes* are also effective in protecting the supply system. Of all these the hydraulic relief valve is to be preferred; the by-pass incurs a considerable waste of water which, in high-head developments, needs to be conserved, and the stand-pipe will, as a rule, entail considerable expense.



ARTICLE 93. *Electric Equipment; Magneto-dynamic Theory.*—*Electric power* is the measure of the useful dynamic work of magneto-electric energy, which is the product of electro-motive force and electric current. *Electro-motive force* is the expression of stress due to the difference in potentiality of the magnetic flux of separate bodies which are brought within each other's sphere of influence. *The magnetic flux* consists of the magnetic waves streaming from and surrounding the source and seat of magnetism; the sphere of this flux is *the magnetic field.*

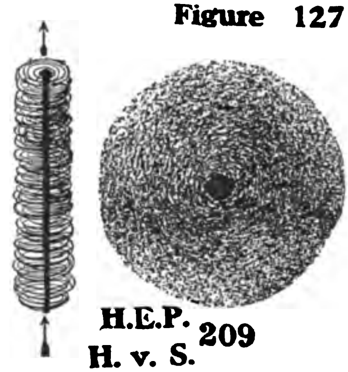
Fig. 126 represents the field of a magnet; it shows the streaming forth of the magnetic flux from one pole, the north, spreading out over more or less space around the body of the magnet, *the core*, returning reassembled into the other pole, the south, and, presumably, passing through the core back to the north pole. If the two poles are connected by a piece of metal, *the keeper*, no evidence of the flux can be traced.

Alessandro Volta, an Italian physicist (1745–1827), and Luigi Galvani, of Bologna (1737–1798), jointly discovered that two dissimilar metals, or a metal and a metalloid, are capable of forming an *electric source* when dipped into an electrolyte, or will produce a difference of electrical potential by mere contact. This is *voltaic electricity.* Many substances become electrified by friction, from which indeed the name, from *electrum*

or amber, a material especially susceptible to electrification by friction. Electricity thus created may be *conducted* through proper mediums—copper, silver, aluminum—and becomes the *electric current.*

Fig. 127 represents a conductor carrying an electric current, by which it becomes the source and core of a magnetic field surrounding it in a manner analogous to that of the magnet shown in Fig. 126, the flux taking the general shape and characteristic of circular whirls; in fact the electricity or current appears to reside largely in this flux, which, upon a rupture of the conductor, retreats into the core and manifests itself by the spark which escapes at the point of the conductor's break.

Figure 127



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H. v. S.

When a conductor, carrying an electric current, is wound about a piece of magnetizable metal, the latter becomes a magnet and displays all the characteristics above described; or, if such a piece of metal is pushed into the interior of a conducting coil carrying an electric current, the piece likewise becomes a magnet. From these phenomena it is deduced that the molecules of certain matter, especially metals, are individual magnets resting normally in an unharmonious state, that is their pole directions are not continuous, and that the influence of the electric current, and the accompanying magneto-electric flux, orders these molecules into a complete chain of magnets, thus opening the way for the flow of magnetic lines through the matter and, therefore, the appearance of the magnetic flux around and about the magnet. Michael Faraday, an English physicist (1791–1867), discovered that the moving of a conductor through a magnetic field and crossing the lines of force induces *electro-motive force* in the conductor in direction at right angles to that of the conductor's motion and to the lines of force cut by it.

This is the basic principle of electric generators and motors,—that is, converting mechanical energy into electric current, or vice versa, by means of magneto-electric energy induced by the moving, generally rotary, of a system of electric conductors through magnetic fields.

Magneto-electric Units.—The flux lines passing through the air space of a magnetic field are the *lines of force,* the magneto-electric energy

finds its origin in the influence of these lines, and the force they represent upon bodies susceptible to such influence and which cross their paths, their aggregate, *the magnetic flux*, is denoted by "N," being the measure of the total number of lines of force, which, as appears from Fig. 126, is the same along any direction across their path. The lines passing through the magnet proper are the *magnetic lines*, which are measured by the number in a unit cross-sectional area, one square centimetre, of the core, and are denoted by "B" or *flux density*. The magneto-motive force emanating from the magnetic flux is denoted by "H," and is measured by the C. G. S. (centimetre-gramme-second) work unit, *the dyne*, a force which imparts a velocity of one centimetre in one second to a mass of one gramme; a unit magnetic pole is one which repels a pole of equal strength, at a distance from it of one centimetre and in air, with the force of one dyne; a field of unit intensity exists therefore at a distance of one centimetre from a unit magnetic pole. The functions relating to the generating and conducting of electric currents are somewhat analogous to those concerned with the flow of water, pressure, volume, and friction of conduit being typified by (pressure) electro-motive force, (volume) current, and (friction) resistance in the conductor.

The unit measure of electro-motive force, or electric pressure, is *the volt* (after Volta), which represents that force induced in a conductor cutting 100,000,000 lines of force in one second; its symbol is E. M. F. or "E."

The unit measure of current is *the ampère*, named so by the International Electrical Congress at Paris in 1881, in honor of the celebrated French electrician André Marie Ampère; this unit represents a rate of flow, or transmission of electricity, which will pass, with the pressure of one volt, through a conductor whose resistance is unity; it is symbolized by "C."

For the resistance unit *the ohm* was adopted by the International Electric Congress in 1893, in honor of Dr. G. S. Ohm; this represents such a resistance of the conductor as will limit the flow of electricity under pressure of one volt to a current of one ampère; its sign is "R."

Therefore $\quad E = C \times R, \qquad C = E \div R, \qquad R = E \div C.$

The electric energy or power unit is *the watt*, named in honor of the Scottish engineer and inventor James Watt (1736–1819), which represents

that rate of work resulting from unit current flowing under unit pressure, therefore also called the volt-ampère.

One watt equals 1 ÷ 746 horse-power, or
one electric horse-power, Ehp = 746 watts and
1000 watts are one kilowatt.

The electro-motive force represents the origin of the final output, its advent through induction, just as the head in hydraulics renders the flow, or the current, available; and, since its magnitude is proportional to the number of force lines cut in a given time, it is evident that it depends upon the three functions, force lines, conductor, and movement.

"N," the number of force lines cut in one second, depends upon the section, the magnetic permeability, and the magnetization of the magnet. The section may be any which adapts itself to the purpose; generally speaking, it is a good feature of a dynamo to have the field magnet or magnets of super-large section; the field may be that of several magnets of suitable form and conveniently placed for the economic and efficient design of the machine. The magnetic permeability of the materials used for dynamo magnets differs considerably; wrought iron probably is of the highest degree, followed by cast-iron and mild steel. The magnetization may be to any degree within the limit of saturation; it is provided by passing conductors around the magnet core, and its measure is expressed in *ampère-turns*, which represents the unit of magneto-motive force and is equal to that produced by a current of one ampère flowing around a single turn or spiral of the conductor and is denoted by "C S."

"Z," the second function of electro-motive force, is the number of conductors which cut the lines of force; they may be as numerous as the machine's arrangement will allow; Z represents the number of conductors connected in series.

The third function, "n," relates to the motion or movement of the conductor, and therefore is a question of speed and of mechanical consideration chiefly.

The theoretical magnitude of the electro-motive force is expressed by

$$E = n \times Z \times N \div 100{,}000{,}000, \text{ or } = n \times Z \times N \div 10^8.$$

ARTICLE 94. *Some Current Symptoms.—Alternations.*—As the electromotive force, and therefore the current, finds its origin in magnetic

stresses, it follows that it will fluctuate in magnitude and change in direction as the number of force lines traversed by the conductor increases or decreases, and that therefore the generated current *alternates*.

Fig. 128 illustrates the simplest method of current generating by one conductor, being a loop, revolving axially between the two pole faces of a magnet. Reference to Fig. 126 of the general grouping of the lines of force makes it clear that when the loop is in the vertical position it embraces the least number of force lines; while turning through a right angle the number of lines cut by the conductor gradually increases, and they become greatest when the loop reaches the horizontal position; during this path the electro-motive force has constantly risen and has maintained the same direction. Descending through the second quarter turn, the number of force lines cut by the conductor decreases until the vertical position is reached and with it the low point of magnitude. Rising through the third and fourth quarters the same process is repeated.

Figure 128

H.E.P. 210
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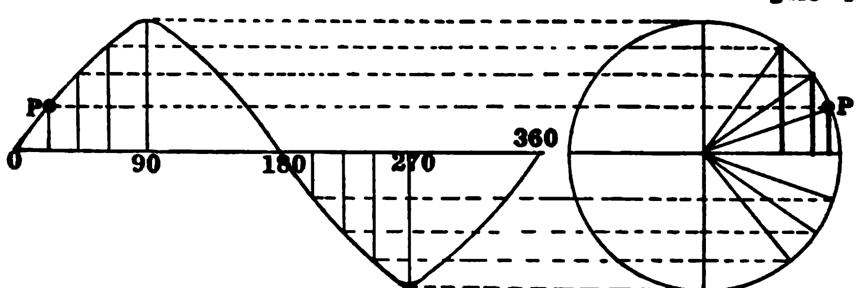
Figure 129



H.E.P. 211
H. v. S.

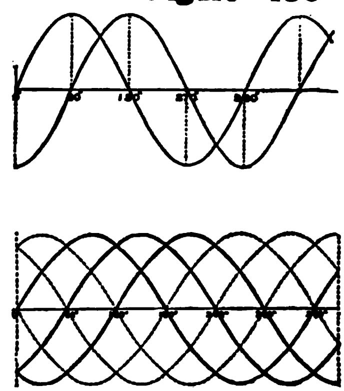
Fig. 129 shows diagrammatically the linear development of these alternations of the current in magnitude and direction during one complete revolution of the conductor; the horizontal line corresponds to the vertical position of the conductor as above described, the upper and lower points typify the horizontal position of the conductor. The

movement of any fixed point, "P," may be traced around the circumference of a circle the radius of which corresponds to the amplitude of the highest point which is reached by the current's magnitude, and from this it is apparent that the electro-motive force at any point, or at any period of time, in the path of the alternating current equals the sine of the angle through which the conductor has turned from its initial vertical position.

Frequency.—The current wave due to one complete revolution of the conductor is called *a period*, and the number of periods occurring in one second are denominated the *periodicity* or the *frequency*, or the *cycle* of alternations, and are symbolized by "n." The frequencies equal the product of conductor revolutions per second and the number of the field magnets passed by the conductor during one complete revolution.

Example.—With a speed of conductors of 240 revolutions per minute, or 6 per second, a six-pole (3 magnets) dynamo will generate current of 18 frequencies.

Figure 130



H.E.P. 212
H. v. S.

Phase.—When, as in Fig. 128, one conductor is provided for each magnet, being a group of conductors for each pole, the alternations of the current are *monophase*, —that is, there is one phase only in the complete period; if, however, the number of conductors is doubled, a *two-phase* current results, and *three-phase* when six groups of conductors are provided.

Fig. 130 shows the two- and three-phase current waves, the alternations in the first being of quarter periods, or the phase angle 90°, while in the three-phase current the alternations differ by 60°. Any alternating current other than single-phase is called *polyphase.*

Inductance, Self-induction, Reaction.—As has been shown in Fig. 127, a conductor carrying an electric current is surrounded by a magnetic field, and, according to the general theory of the origin of electro-motive force being due to magnetic stress, it is evident that the current carried by the conductor will, when brought into the sphere of another magnetic field, undergo changes corresponding to the relative stress conditions.

Such an influence upon a conductor is called *inductance*, and manifests itself in destroying the harmonious flow of volts and ampères by the *reaction* of the self-induced electro-motive force and its retardation of the current's phase, the ampères, which fall behind the volts, this condition being called *the lag;* the extent of this disturbance is expressed by the *angle of lag*, which is the measure of that portion of the angle of phase represented by the lagging of the ampère behind the volt wave.

Impedance.—Inductance must be overcome by increased electromotive force,—that is, whereas the normal expression for "C" is E ÷ R, E being the impressed voltage and R the resistance of the circuit, in the presence of inductance C becomes, as illustrated in Fig. 131,

$$C = E\sqrt{R^2 + C^2 L},$$

in which L is a coefficient of self-induction. E of this value is the *impedance*, the impressed voltage or the ratio of produced volts to ampères. When there is no induction, that is in steady currents, impedance equals resistance.

Capacity, Reactance.—The opposite effect of inductance occurs in a current when charging a condenser; this causes the ampères *to lead* the volt phase, which condition is termed *capacity*, being caused by reactance of the condenser.

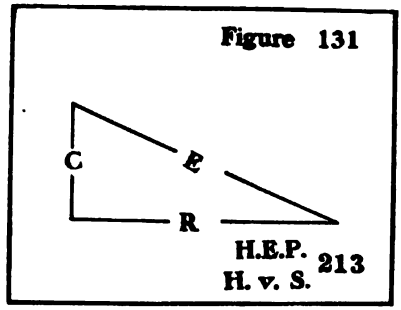
Inductance and capacity, being opposite in effect, may be created for the purpose of counteracting each other, when the current is said to be *non-inductive*, and in that event C = E ÷ R.

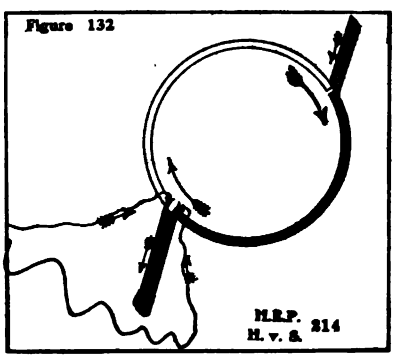
Watt-less Current.—When the phase of volts and current differs considerably by reason of lag or lead caused by self-induction or capacity, the actual watt output is less than what would be indicated by the product E × C. Fig. 131 shows the two components, and these may be considered as representative of the working and the watt-less magnitudes.

Continuous Current; Commutator.—The alternate current (Fig. 132) may be made continuous in direction by connecting the terminals of the conductor to separate segments of a ring, the segments being insulated from each other, and by passing metallic brushes over the circular segment surfaces in such a manner that the gap between the two segments is closed by the brush contact at the period when the alternation of the current reversal takes place; the exterior circuit is connected to the brushes and the commuted current flows into it in continuous direction.

The apparatus through which the alternate current is changed into continuous current is called the *commutator*.

ARTICLE 95. *Dynamo Parts; their Purpose and Design.* — Electric dynamos serve the purpose of generating electric energy, the origin of which may be continuous or alternating current; electric energy may therefore be generated by continuous or alternating current dynamos commonly called D. C. generators and alternators. Any dynamo consists in general of the same parts, — the magnets, which are assembled in the field, and the conductors, which are grouped in an armature; one of these parts revolves, and then is the interior, the other is fixed, and is the exterior; both are concentric of each other. The fixed or exterior part becomes also the frame of the machine, terminating below in a base and frequently having connected to it shaft-bearing stands; the interior or revolving part is connected to the shaft. The variations of types and forms of parts are manifold, and it is not the purpose here to describe any particular make, but to give such an outline of the purpose and theoretical design of the principal parts as to enable the investigator to recognize compliance with or any gross departure from these essentials.





Continuous-Current Dynamos. — *The field* consists of the magnets and their windings. *The magnets* are radial arms or shoes connected to a *yoke;* the extreme ends of the magnet shoes are the *poles*.

In dynamos of large output, such as are employed in the equipment of hydro-electric plants, the field is multipolar, as distinguished from that of small continuous-current dynamos, in which the field consists of one magnet or two poles, being termed bipolar; the multipolar dynamos have four, six, eight, or more poles arranged along the interior periphery of the fixed field, the frame part becoming the yoke, or to the exterior periphery of the rotating field in which the hub represents the yoke.

Magnet arms are generally built up of laminated (insulated) metal disks of wrought iron or mild steel, being of circular or other shape. The field of all large dynamos consists of electro-magnets,—that is, the magnetization is created by passing an electric current through conductors wound around the magnet limbs; this is called *excitation* of the field. The field of a continuous-current dynamo is *self-excited;* the electric current, in other words, carried by the field conductors is generated in the dynamo armature; this is not the case in alternators, as will be seen later on. The field winding may be of the continued armature conductors, which is called *series wound;* or it may be of a smaller conductor than the armature coils and connected with the armature winding as a sort of splice; this is called the *shunt-wound field;* and finally the field winding may consist of a combination of the series and shunt method, the latter overlying the former, and this is called a *compound wound field.* As will be seen later, the field excitation, and therefore its winding, forms one of the most important means of regulation of the output, and therefore this characteristic of the field winding practically conveys, in a large sense, a conception of the efficiency of the dynamo as to the regularity of its output. There are many reasons for the desirability of the compound winding of the field, and it is practically the general practice with dynamos of large output.

The section of the magnets and the amount of the winding depend upon the permeability of the material of which the magnet is formed and the required magnetization of the field, that is of the flux density as expressed by "N" in the formula of electro-motive force, $E = n\,Z\,N \div 10^8$.

The winding is measured by *ampère-turns,* which represent the product of the current, in ampères, carried by the wire and the number of turns around the magnet limb.

The permeability of magnets of wrought iron and cast iron is given in the following table, which is according to Prof. Sylvan B. Thompson, where

H is magnetic force, or the number of lines per square inch of the field;

B is the flux density, the number of lines per square inch of the magnet section; and

u is the magnetic permeability, or the specific conductivity, for magnetic lines, of the material in the magnet.

TABLE 30.

Wrought Iron.			Cast Iron.		
B	H	u	B	H	u
30,000	10.2	2926	25,000	30	843
40,000	14	2857	30,000	53.5	445
50,000	20.9	2392	40,000	163	245
60,000	27.7	2166	50,000	447	112
70,000	40	1750	60,000	940	64
80,000	63	1368	70,000	1750	40
90,000	105	856			
100,000	245	407			
110,000	686	161			
120,000	1850	64			
130,000	4500	28			
140,000	7630	18			

The required ampere turns for the field winding are obtained from different formulas; the result of the following equation, while not exact, will give practical useful values:

$$S\,C = (B \times L \div 2.02) \times 1.25, \text{ where}$$

S C are ampère turns (current in spirals),
L is the length of the air gap in inches, and
2.02 is the number of ampère turns required to produce unit flux density in an air space one inch long,
1.25 is a coefficient to compensate for the drop of magnetic potential.

It may also be stated, as a general excitation rule, that dynamos up to 200 kilowatts output require exciting currents of 0.05 to 0.025 of their ampère output, and of 1000 kilowatts, 0.015 and less.

The magnet section may be determined from N ÷ B; N is the total number of lines of force and B the flux density as per Table 30. It is common practice to make the magnet section 1.66 of the armature core of ring and 1.25 of drum type.

The size of the field magnet conductor is determined from the ampère turns, available winding space, and considerations of the conductor's resistance; the usually adopted density of field-coil current is limited to 2000 ampères per square inch. A convenient formula for the finding of field coil wire is

$S\,C = e \div r$, where
S C is ampère turns;
e, the voltage at the terminals of the field coils; and
r, the resistance of one turn of the wire.

Resistance in copper wire is as per the following table.

TABLE 31.—TABLE OF COPPER WIRE, SIZE, DIMENSION, WEIGHT, AND RESISTANCE.

Gauge B. & S. Brown & Sharpe	Mils diam.	Circular mils c.m. = d^2.	Resistance in ohms per M ft.	Weight in lbs. per 1000 feet bare.
0000	460	211,600	0.04904	640
000	410	167,800	0.06184	508
00	365	133,100	0.07797	403
0	325	105,600	0.09827	320
1	289	83,690	0.12398	253
2	258	66,370	0.15633	201
3	229	52,630	0.19714	159
4	204	41,740	0.24858	126
5	182	33,100	0.31346	121
6	162	26,250	0.39528	99
7	144.3	20,736	0.491	63
8	128.5	16,384	0.6214	50
9	114.4	12,966	0.7834	39
10	101.9	10,404	0.9785	32
11	90.74	8,281	1.229	25
12	80.81	6,561	1.552	20
13	71.96	5,184	1.964	15.7
14	64.08	4,096	2.485	12.4
15	57.07	3,249	3.133	9.8

Wire Formulas.—c.m. is circular mils; R is resistance; W is weight; L is length.

$R = 11 \times L \div c.m.$; $c.m. = 11 \times L \div R$; $W = L^2 \div 30{,}000\ R$; $L = c.m. \times R \div 11$; $R = L^2 \div 30{,}000\ W$; $c.m. = 1{,}000{,}000\ W \div 3.03\ L$; $L = W \times R \times 30{,}000$.

The armature consists of the core and the conductor. The cores may be of the ring or drum shape; they are built up of laminated sheet iron or steel disks, commonly from 25 to 50 mils in thickness, the ratio of external to internal diameter of ring disks being 5:3. Drum armature cores are generally greater in diameter than they are long. The exterior disk perimeter is shaped for the convenient and safe placing of the conductors, the forms for conductor seats, or beds, being generally of the tooth or slot type; sometimes they are circular holes near the exterior periphery.

The armature winding consists usually of closed rectangular copper bar coils; the schemes of windings are of many different kinds,—parallel, series, or mixed groupings,—and in either of them the winding may be

of lap or wave. *The number of armature conductors* is ascertained from Z in $E = n Z N \div 10^8$; E is the sum of the volts required for the exterior circuit and those lost in the armature; the latter are expressed by $r_a C_a$, where r_a is the armature resistance and C_a the armature current. *The size* of the armature *conductors* is found from the wire table, the practice being to allow a current density of 3000 to 4000 ampères per square inch of conductor. *The core section* is found from the determination of the winding and size of the conductors.

The commutator, sometimes called the *collector*, consists of bars and brushes. *The commutator bars* are usually of copper drop forgings and of a length equal to 1.2 inches per 100 ampères; their number is proportionate to that of the number of armature coils. *Armature brushes* are of woven copper wire gauze, or of spring copper strips, or they may be of carbons, but the use of the latter requires longer commutator bars. Mica is most generally used for the insulation of the commutator bars from the armature hub and from each other. *Brush holders* are the device by which commutator brushes are maintained in position; they are of various designs.

Alternators for large output, as required for the equipment of hydro-electric plants, are generally polyphase, of 25 to 60 cycles and of high voltage; owing to the greater economy and safety with which insulation for high voltage can be secured in a stationary rather than a moving mechanism, the high-voltage alternators for transmission service are of the revolving-field type, the armature being the stationary or fixed part.

The field is of the drum type, the magnet yoke being the hub and core, the limbs are secured to it radially; the poles are more numerous than in the continuous-current dynamo. The magnets are of like material and construction as described for the D. C. generators, but the flux density of the magnets is taken at a considerably lower constant, approximately 42,000.

Excitation is not practicable by the alternating current, and therefore is generally provided from a separate continuous-current dynamo; a larger inductive drop, due to the choking in armature coils and the demagnetization effect of the armature current, must be allowed than in the D. C. dynamo in determining the ampère turns of the field winding which is always compounded.

The relation of speed, number of poles and of frequency has already been noted; it is expressed by $n = 0.5\,p \times (N \div 60)$, where n is the

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frequency of alternations, the cycles or periodicity, p is the number of poles (not of magnets), and N the revolutions per minute, which is the usual measure of speed.

The armature of the revolving-field alternator is a cast-iron ring into which the laminated armature disks are dovetailed along the interior periphery; armature disks are somewhat thinner than those used in continuous-current dynamo armatures because of the higher voltage, usually from 0.012 to 0.008 inch. *The armature winding* is determined in like manner as that for D. C. generators, with proper consideration of the difference between the impressed and the actual volts which was presented in Article 94; the lap winding is preferable for high voltage.

ARTICLE 96. *Current Reorganization.*—The generated current is, as has been shown in Article 94, continuous or alternating, and the latter is monophase or polyphase; to change the generated current into any other kind of current, for the purpose of service, is characterized as reorganization of the current.

The generated continuous current may be changed to alternate current by the aid of a *motor-generator*, being a composite machine consisting of a motor supplied and operated by the generated continuous current, which in turn operates an alternator connected to it, its output being alternating current of mono- or polyphase. The generated alternating current may be rectified into continuous current by means of the same device, in which case the motor, of the induction type, is supplied with and operated by the generated alternating current and itself operates the continuous-current dynamo.

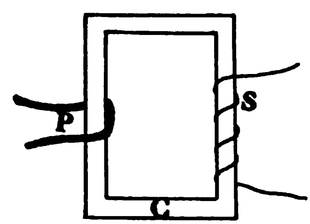
The same result may be obtained through the agency of a *rotary convertor*, and by a shorter process, this machine being a continuous-current dynamo fitted with *collecting rings* in addition to the commutator. The alternate current passes by way of these rings into the armature coils and thence through the rotary and its commutator, and therefore issues as continuous current. The general design of these reorganizing machines is similar to that of generators, as presented in Article 95.

The phase of the current may be changed from mono- to polyphase, and *vice versa*, by the aid of a *phase transformer*, being of the type of a motor generator, in which the phase reorganization is brought about by the addition to the commutator of *slip rings* and by their appropriate connection to the armature conductors.

Article 97. *Current Transformation.*—When the voltage (E. M. F.) of any current is raised or lowered, it is said to be *transformed.* As may be inferred from the origin and characteristics of continuous current, as briefly presented in Article 94, this type of current can be transformed only by means of moving apparatus.

Continuous-current dynamos may be connected in series, in which case the voltage in the exterior circuit is the aggregate of the voltages of the D. C. dynamos thus connected; the only other process of transforming continuous current is by the agency of the *motor generator* mentioned in the previous article.

Fig. 133



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Alternating-current voltage may be raised or lowered by the aid of *static transformers*, in accordance with the following theory. It has been noted in Article 93 that when a conductor carrying current is passed around a closed magnetizable core, magneto-electric force is set up in the core; when the current in the coil is of the alternating kind, the induced magnetic force retains the characteristics of alternations both in direction and flux density, and by virtue of this condition magnetic stress is set up which becomes the source of an electro-motive force other than that of the current in the circuit which passes around the core. This induced electro-motive force acts in opposition to that of the magnetizing circuit and thus incites the latter to increased activity. If another conductor circuit is passed around the same core and is not connected with the magnetizing circuit, the increase of E. M. F. in the latter finds an outlet, or overflow so to speak, into this other conductor, and, under the constant pressure from its source, and the continued conflict between it and the back E. M. F. due to the magnetic stress set up in the core, it continues to pass into the second conductor.

This is the principle of alternating current transformation illustrated by Fig. 133, in which C is the core above mentioned, P is the magnetizing circuit called *the primary*, and S is the second circuit, being *the secondary.*

It may be pointed out that this current phenomenon may be likened to what transpires in the operation of a generator, that the primary is like the field and the secondary the armature, while mechanical motion is represented by the alternations, that is the continuous coming and going of flux density and direction, which is practically of the same effect, as far as cutting of magnetic lines in a space of time is concerned, as when the armature revolves and passes through the field in that manner; and from this it will at once be clear that no such phenomenon can occur with continuous current that, though the core would be magnetized, there would be no current in the secondary.

Static Transformer Analysis.—When the secondary is an open circuit, there is no transformation of E. M. F.; the magnetization of the core originates a back electro-motive force of practically the same value as that of the primary, being diminished only by the voltage required to overcome the resistance of the primary coil.

When the secondary is closed, a current is set up in it opposing that in the primary, its first effort being to demagnetize the core, and, since it is the expressed purpose of the primary to magnetize the core, a conflict or stress is set up which calls forth greater efforts of the primary to overcome the opposition of the demagnetizing influences, with the result that a distinct current passes into the secondary, being of like frequency and other characteristics as the primary excepting as to the E. M. F., as it is the purpose of the process to alter or transform this by raising or lowering the voltage of the primary to any desired in the secondary. The E. M. F. in these two circuits is proportional to the number of turns in each which pass around the core, which is analogous to the principle of field excitation explained in Article 95. In Fig. 133 the primary is shown of one turn while the secondary has four turns around the core; the voltage of the secondary will therefore be four times that of the primary, while the current, ampères, in the secondary will be one-fourth of those of the primary. In this case the voltage is raised by a *step-up transformer*, the lowering of the voltage would be secured through a *step-down transformer*, both being of the same design and construction excepting as to the respective winding of the primary and secondary coils, which represents the *ratio of transformation*. In this connection it should be noted that with like current density per conductor section unit (square inch) the ratio of transformation must be accompanied by

a corresponding adaptation of conductor cross-section; thus the conductor in the secondary of a step-down transformer must have an increased copper section over that of the primary which is inversely to the voltage drop.

Static transformers consist of the core, the windings, and the shell. The cores are of many different shapes, being generally formed of laminated iron disks; the windings are of insulated copper wire. There is no particular limit to the transformation ratio, the principal consideration in this regard being the heat which is generated in the transformer and may rise to a degree at which the insulations would suffer destruction; high-voltage transformers are therefore especially designed to keep this feature well insured by employing different methods of cooling the transformer and its various parts. Transformer coils are often placed in oil, which receives the heat and transmits it to the shell where it is readily radiated; the oil is also separately cooled by passing water through pipe coils immersed in it; again cooling is effected by passing cold air through the transformers, either by a blower or fan system.

The losses in static transformers may be kept very low by proper design and construction; they are due to the resistance of the secondary coils, to magnetic flux friction in the core, and to idle or *eddy currents* which are set up in the core; the aggregate need not exceed 2 per cent.

ARTICLE 98. *Current Transmission.—Theory.*—Electric energy may be conducted to any distance, as its flow will always continue from a high to a lower potential, but, as in transmission of energy of any form, work is constantly being performed during its passage through the conductor, and, at some point or other, the energy thus expended may become so great a part of the impressed volume that its transmission, under such conditions, represents a waste rather than a gain.

The work of the current during its transmission is that of overcoming the conductor's resistance to its free passage; just as the flow of water through a pipe is impeded by the roughness of the pipe's perimeter or its change of section, which is overcome by the expenditure of head, so the transmission of electric energy is made possible only by the overcoming of the conductor's resistance, which is accomplished by the expenditure of E. M. F. By Ohm's law $C = E \div R$ and $E = C R$ and

$R = E \div C$, from which R, the resistance in any length of a certain conductor, may be ascertained from $l \times r \div cm$, where

l is the length of the conductor in feet;
r is the resistance unit, being the resistance of one foot of copper wire of one mil section (one mil foot), which = 10.8 ohms, but is taken as 11 ohms, whereby the irregularities of the wire's section and the faults of joints are compensated for;
cm stands for the area of the conductor's section in circular mils; therefore
$R = 11 \times l \div cm$, and, inserting this in above equation for R,
$R = E \div C$, it becomes $11 \times l \div cm = E \div C$, in which
C is the current (ampères) which is to be delivered for service,
E is the electro-motive force to be expended in overcoming the resistance of the conductor, being termed the transmission *loss* or the *voltage drop*.

Aluminum wire is well adapted for the purpose of transmission conductors; the ohmic resistance coefficient of aluminum is 17 per mil foot instead of 11, the coefficient for copper, and this value must be inserted in above formula when aluminum wire is being investigated instead of copper wire.

Since, from above formula, the resistance may be found for any size of conductor, so may the size be determined from it to transmit a fixed current at a given line voltage over a known distance; that is, from above

$$cm = l \times 11 \times C \div E \text{ for copper and}$$
$$cm = l \times 17 \times C \div E \text{ for aluminum.}$$

The weight of 1000 feet of 1000 cm copper wire is approximately 3 lbs. The weight of 1000 feet of 1000 cm aluminum wire is approximately 1.5 lbs. By symbolizing 1000 feet of the conductor's length as L, the weight of the conductor may be expressed from the foregoing formula by

$$W = L^2 \times 33 \times C \div E \text{ for copper conductors, and}$$
$$W = L^2 \times 23.5 \times C \div E \text{ for aluminum conductors.}$$

This refers to a single conductor.

From these expressions and from formerly discussed current characteristics, it is apparent that E is the principal factor controlling the value of W, for instance when the ratio of percentage of line drop from the impressed electro-motive force is fixed, the doubling of the impressed voltage will halve C, double E, and reduce W to one-fourth; therefore, *the weight* of the transmission conductor for given length, current, and drop *varies inversely as the square of the impressed voltage,* and for given current and drop and impressed voltage in direct proportion to length of transmission, *the weight remains constant;* while for given current and drop the *weight increases as the square of the* length.

For the purpose of hydro-electric practice all current transmission factors may be determined from these formulæ with sufficient accuracy to estimate the transmission conductors and their cost, but many refinements enter the problem when considered from a scientific stand-point. Diagram 16, appearing in connection with Article 28 in Part I. of this book, has been calculated for copper wire from above formula and is correct, within its scope, for the purpose of estimating the wire quantity.

Transmission of continuous and of single and two-phase alternating currents is by two wires, all the current passing over every part, while three-phase alternating current may be transmitted by three conductors, each of which carries the current of its particular phase. In the above formulæ, for determining cm or W of transmission-line conductors, the length, when considering continuous and single and two-phase alternating currents, or in two-conductor circuits, is the developed length of the conductors or double the transmission distance, while for three-phase alternating current transmission the length is equal to the transmission distance. In finding the weight, therefore, L is multiplied by four for two-conductor lines and by three for three-conductor lines; for this reason a three-conductor transmission line (of three-phase current) requires only 75 per cent. of the weight of copper needed in a two-conductor line of the same energy.

Of the current symptoms briefly outlined in Article 94 those specially applying to current transmission are: *inductance,* which is the absorption of electric energy while producing a magnetic field around the conductor, or the setting up of an electro-motive force opposite to that impressed in the alternate current conductor, resulting in voltage drop at the line terminal; *capacity,* being the reactance of the transformation of alter-

nating current and resulting in an increase of current in the circuit; and *resonance*, which is the neutralization of inductance and capacity with the effect of a considerable rise in the line voltage. The effects of all these in transmission are best met by an addition of not less than five per cent. to the cm or W of the transmission conductor as found from the quoted formulæ.

ARTICLE 99. *Current Regulation.*—This subject, like the preceding electrical topics, will be treated herein only to the extent of presenting the apparatus and the purpose which it serves, as it may have to be considered in connection with the planning of the hydro-electric operating plant and its equipment and for the preparing of the estimate covering these.

Regulation of current generating is, as has been pointed out in Articles 94 and 95, first secured by the turbine governor and second by field excitation, the first maintaining, as near as practicable, constancy of mechanical speed, the second of electro-motive force or pressure. The former is practically automatic, but the latter needs more or less personal attention, depending upon the current reorganization and transformation methods and means, the character of the current service, and the fluctuations of the loads. At any rate certain apparatus is required to indicate output characteristics and detect irregularities, while others are needed to furnish ready correction for these.

The purpose of regulation is generally to maintain constancy of pressure, voltage, in the exterior circuit, which, in the case of hydro-electric plant output, is the transmission line. At the generator, as has been noted, this is principally accomplished by regulating the exciting current, as this is the origin of the electro-motive force; on the line, regulation may be secured through static transformers with alternating current. Transformation is generally effected through *a bank of* transformers of equal units connected in series, but, as the cause of the fluctuations is mainly to be found at the service end of the line, regulation is best arranged for at this terminal, the substation.

Regulating apparatus may be classified into switches, testing instruments, and correctors, most of which are collected on the *switchboard*, which consist of marble or slate tablets or *panels* placed against a suitable frame in the generating room and in the substation. The circuit conductors are led to the back of the switchboard panels, the latter being

perforated at the places where the switches or apparatus are to be secured so that they can be connected with the conductor circuits.

Switches, or circuit breakers and closers, are devices by which conductor circuits are closed or opened; they are of various designs, all aiming *to make or break* the current by avoiding any arcing across. High-voltage switches are therefore necessarily of special designs and construction, and wherever practicable the switching of such currents is arranged for on the low-voltage side of the circuit. Circuit breakers may be of air or *oil break* type, the latter being always employed in high-voltage practice, and, when the voltage is very high, 20,000 and more, oil circuit breakers are operated by secondary pneumatic or electric power devices and are removed from the vicinity of any of the other electric equipment.

Testing instruments comprise *ammeters* (ampèremeters), which are of the galvanometer type, indicating the current strength by the deflection of a magnetic needle placed inside or over a coil of insulated wire through which the current to be measured is passed. *Voltmeter* is an instrument similar to the ammeter, of electro-magnetic type, its purpose being to indicate the voltage of the current.

Wattmeter is the third of this class of testing instruments, indicating the watts; though voltage and ampères are indicative of the watt output of the continuous current, this is not necessarily the case with the alternating current, where the product of the volts and amps represents the *apparent watts*, while the wattmeter indicates the *effective watts*.

Phasemeters indicate the *power factor* of the current, which is the ratio of effective and apparent watts, or the *phase difference*. *Synchronizers or phase indicators*, as their name implies, indicate when synchronism, that is union of frequencies in the alternating current, exists between different alternators which are to be operated in parallel.

Pilot lamps are connected across the dynamo terminals for the ready indication of the approximate pressure by the degree of their incandescence.

Ground detectors are required for detecting and measuring grounds and leaks.

Correctors are *lightning arresters*, which may be placed at the generating plant, along the transmission line, and at the substation, for the purpose of diverting lightning charges. There are various types of these;

one in common use consists of a number of air gaps between conductor plugs set in a porcelain block. *Guard wires* are also employed for lightning diversion from transmission lines; they are metallic circuits (iron) grounded at intervals of 1000 feet.

Choking coils are of copper wire so wound on or around iron core pieces as to possess high self-induction when used on alternating circuits; their purpose is to obstruct or *cut off* alternating current with a smaller loss of energy than if it were used as ohmic resistance.

Booster is a dynamo connected to a separate circuit for the purpose of raising its voltage above that of the other system of which it forms a part.

Rheostats are adjustable resistance coils.

ARTICLE 100. *Electric Generating Plant.*—The character of the generating equipment is to be determined with a view of securing high efficiency of resourceful output at reasonable first and lowest practicable maintenance, depreciation, and operating cost.

The matters to be determined are:

(a) the kind of current to be generated,
(b) the generator voltage,
(c) the generator units,
(d) the mechanical power application to generators and exciters,
(e) the types of generators, exciters, and governors,
(f) the arrangement of the apparatus;

the controlling features in this quest are:

(g) the distance to the current market,
(h) the character of the current service,
(i) the available hydraulic and mechanical factors, and
(j) the output efficiency, time delivery, and cost of apparatus.

The current to be generated must be determined from the character of the current service and from the market distance. Continuous current is largely employed for arc lighting, exclusively for electrolithic operations and accumulator charging, and is preferable, at the present state of the practice, for power service in which the fluctuations of the load are sudden and excessive, such as electric traction and lifting. The alternating current is largely used for incandescent lighting and mechanical

power application for factory and shop machinery and tools, pumps and elevators.

The influence of the distance to the market is most potent with a transmission system, and, as far as it has a bearing upon the choice of the kind of current to be generated, it is largely a question of economics. The principal items in the cost of the transmission plant are the conductors, and the weight of these, as has been noted, with fixed output, distance and line drop, depends entirely upon the impressed or generating voltage. The voltage of continuous-current generators is limited, by reason of the commutator device, to approximately 2000 volts; if several D. C. generators are connected in series the voltage sent into the exterior circuit will be the aggregate of all the generators thus connected. Alternators may be designed to generate at high voltages, 200,000 being now altogether practicable. Transformation of continuous-current voltage is practicable only by the aid of motor generators, while that of alternating current is accomplished through static transformers. All these limitations of the continuous current point to the alternating current as preferable when transmission to a considerable distance is required, even though the service calls for continuous current only.

Example 1.—An electrolithic plant is to be furnished 1000 kilowatts of continuous current at a voltage of 250 from a hydro-electric power plant one mile distant; the drop or loss of pressure in transmission is to be kept within 5 per cent. If continuous current is generated at a voltage of 275 and transmitted at this voltage to the electrolithic plant, the generating plant will consist of one or two D. C. generators and the transmission line; the conductors for the latter are two strands of wire of aggregate weight found from $W = 4\,L^2 \times 33 \times C \div E$.

L is in 1000 feet units, the length of a transmission mile of conductors being taken at 5400 feet to compensate for the sag between supports;

C is the current in ampères to be delivered at the electrolithic plant, being $= 1{,}000{,}000 \div 250 = 4000$;

E is the line voltage drop $= 275 \times 0.05 = 14$; therefore

$W = 156.64 \times 33 \times 4000 \div 14 = 1{,}470{,}857$ lbs.

If alternating current is generated at 2300 volts, which is a standard type alternator, and transmitted at this voltage to the customer in ques-

tion, one mile distant, with a line drop of 5 per cent., the generating plant will consist of one or two alternators, the transmission line, and a rotary converter by which the current is reorganized to the required continuous current. The conductors for this line will consist of three strands of wire, the current being of the three-phase type, and the weight of the line wire is found from $W = 3 \times L^2 \times 33 \times C \div E$, in which

$C = 1{,}000{,}000 \div 2185$ and $E = 2300 \times 0.05$; therefore
$W = 3 \times 29.16 \times 458 \div 115 = 11{,}500$ lbs., to which should be added for induction 5 per cent., making the total weight = 12,075 lbs.

The alternator may generate at a higher voltage without the necessity of raising the line voltage, and thus the weight of the required conductors may be still further reduced; however, it is clear that this plant should generate alternating current.

Example 2.—An electric power crane unloading iron ore and fuel from vessels is to be supplied with 500 kilowatts of continuous current at a voltage of 400 from a hydro-electric power plant half a mile distant. For continuous current the generating plant would consist of one D. C. generator and the transmission line, and the weight of the required conductors is again found from

$$W = 4 \times 2.600^2 \times 33 \times 1259 \div 20 = 55{,}688 \text{ lbs.}$$

If alternating current is generated for this service, the plant will consist of the alternator, the line, and a rotary converter; the current may be generated at a voltage of 2300 and thus transmitted without step-up transformers being added; the line would consist of three strands of wire and the weight from

$$W = 3 \times 2.600^2 \times 33 \times 234 \div 115 = 1364 \text{ lbs.},$$

to which should be added 5 per cent., making the total conductor weight approximately 1432 lbs., being 54,256 lbs. less than required for the continuous-current line. The question to be decided is, the comparative cost of this quantity of conductor and that of the rotary, to which the operating and maintenance cost of the rotary may have to be added

unless the customer will take over its operation. From this it appears that the distance limit at which continuous-current generation, for continuous-current service taking up the entire output, is to be preferred to alternating-current generating and current reorganization, lies inside of the one mile for outputs of less than 1000 kilowatts, and that for greater distances or larger outputs the alternating current proves the more economical for the generating plant.

With the current decided, the next question is as to the *phase* and the *frequency*. A great deal might be said on this topic as having a bearing one way or the other, but, after all, the safest guide may be found in practice, which, in this country at least, at present is largely in favor of the three-phase current at the generating end. The important influence of phase characteristic on the transmission-line conductor has already been noted and distinctly points to the preference of the three-phase current.

The question of frequency is not so positively settled, as it depends largely upon the character of the current service. When the bulk of the current is intended for power service, the lower frequency of 25 cycles lays claim to several advantages over a higher one. The cost of rotary converters rises with the frequency, as it requires more poles and more costly armature construction than for low frequencies; on the other hand, the cost and weight of static transformers is greater for low than for higher frequencies, and therefore as far as the cost item is concerned the most economical mean between transformers and reorganizers should readily offer the solution. When the service load is fairly distributed between lighting and power production, the 60-cycle current appears to be preferable over that of lower frequencies.

The next current characteristic to be decided is the *generator voltage*, and for transmission duty there is little doubt that every desideratum points to the highest practicable. Alternators of high voltage should be less costly than low-voltage generators plus step-up transformers, though their depreciation and maintenance may be somewhat greater. However, the factor of transformer loss, which with high-generator voltage may be avoided, carries considerable weight in the summing up. For transmission distances up to ten miles it will generally be found more economical to generate at a sufficiently high voltage to make step-up transformers unnecessary, and this distance may even be doubled if the alternator voltage comes up correspondingly.

Generator units are more largely to be determined by the hydraulic conditions than any other factor. It will now be fully realized that high generator speed is much to be desired, and if the drive is to be by direct coupling to the turbine shaft the choice of the generator unit will generally not be doubtful. As a rule, it is advisable to have duplicate units of like capacity to meet unforeseen emergencies by which any of them may be put temporarily out of commission, and, with this important proviso and the desideratum of high speed, the units are preferably of the largest practicable output, that is within the limits of economical standard designs.

This leads to the fourth point to be determined in connection with the composition and character of the generating equipment, *the mechanical power application* for generators and exciters. There exists at the present day a certain almost hysterical clamor for direct-connected apparatus; ostensibly it is influenced by the desire to make the greatest showing of economical energy utilization, to avoid the loss due to other than direct drives, while frequently the stickling for this arrangement results in far greater energy waste, because the most suitable generating equipment is barred out, and in this manner a rational study and analysis of the opportunity is overshadowed by the desire to have things "up to date," at least as far as appearances go. There are quite a number of the present important hydro-electric plants which might have been bettered by several per cent. of output if equipment were belt driven instead of being direct connected. At any rate direct-connected apparatus may not always be the most economical (or, rather, efficient) solution of this problem. Speed lies at the root of it all, and by this the size, voltage, and cost of apparatus are fixed; a balancing of the value of the lost energy through belt drive, as compared with direct connection, against the difference in cost of the generating and transmission plant which is required for either programme, will, as a rule, plainly point to the one to be preferred. There is much less to be said in favor of gear-driven equipment, which, however, comes under consideration with low-head developments and may then be the only solution of this question of mechanical power application.

Exciters are preferably driven by separate turbine units. *The type of the generator equipment* should generally be decided in favor of that offered by the lowest competitive tender who guarantees prompt delivery of the specified apparatus. All of the leading American manufacturers of electric dynamos produce equally reliable and efficient machines, and,

when the desired characteristics and output efficiencies are clearly specified, as will be outlined in the last chapter dealing with specifications, this question should be satisfactorily solved by adopting approved business methods.

The arrangement of the equipment in the power station must be made with ample allowance of operating space around every machine and accessory apparatus, good light should be available for switchboards, and the station should be planned not merely to hold the outfit but also to move it about, in and out, without interfering with operations, and, finally, possible additions must be taken into consideration, as a paying plant is also a growing one. Diagram 40 gives the approximate dimensions of alternators of different output and speeds at 2300 volts, which is standard and well adapted to most conditions.

The following table gives a list of generators, their capacity and speed, which are of standard types with American manufacturers, and can therefore be obtained within reasonable delivery periods and on competitive tenders.

TABLE 32.—STANDARD GENERATORS, ALTERNATORS, 2 AND 3 PHASE, 2300 VOLTS AND UP.

Drive.	K.W.	Frequency.	Poles.	Speed.	Drive.	K.W.	Frequency.	Poles.	Speed.
Coupled......	100	69	24	300	Coupled......	225	60	12	600
Coupled......	100	60	8	900	Coupled......	250	60	36	200
Coupled......	105	60	28	257	Coupled......	250	60	32	225
Coupled......	110	30	6	600	Coupled......	250	60	28	257
Coupled......	115	60	8	900	Coupled......	250	60	24	300
Coupled......	125	60	32	225	Coupled......	250	60	16	450
Coupled......	125	60	26	277	Coupled......	250	60	12	600
Coupled......	120	25	6	500	Coupled......	275	60	16	450
Coupled......	120	25	4	750	Coupled......	275	60	12	600
Coupled......	150	60	32	225	Coupled......	300	60	72	100
Coupled......	150	60	26	227	Coupled......	300	60	48	150
Coupled......	150	60	20	360	Coupled......	300	60	36	200
Coupled......	150	60	14	514	Coupled......	300	60	32	225
Coupled......	150	60	12	600	Coupled......	300	60	28	257
Coupled......	150	60	8	900	Coupled......	300	60	24	300
Coupled......	175	60	36	200	Coupled......	300	60	18	400
Coupled......	175	60	12	600	Coupled......	300	60	16	450
Coupled......	180	60	8	900	Coupled......	300	60	12	600
Coupled......	200	60	36	200	Coupled......	325	60	36	200
Coupled......	200	60	32	225	Coupled......	330	50	16	375
Coupled......	200	25	6	500	Coupled......	350	60	16	450
Coupled......	200	50	12	500	Coupled......	350	60	12	600
Coupled......	200	60	14	514	Coupled......	360	25	8	375
Coupled......	200	60	12	600	Coupled......	360	25	6	500

TABLE 32.—STANDARD GENERATORS, ALTERNATORS, 2 AND 3 PHASE, 2300 VOLTS AND UP.—*Continued*.

Drive	K.W.	Frequency.	Poles.	Speed.
Coupled......	360	25	4	750
Coupled......	375	60	30	240
Coupled......	380	60	16	450
Coupled......	380	60	12	600
Coupled......	400	25	20	150
Coupled......	400	60	36	200
Coupled......	400	60	28	257
Coupled......	400	60	24	300
Coupled......	400	60	16	450
Coupled......	420	60	24	300
Coupled......	420	25	18	167
Coupled......	420	25	6	500
Coupled......	425	60	30	240
Coupled......	425	50	16	370
Coupled......	450	60	60	120
Coupled......	450	60	48	150
Coupled......	450	60	40	180
Coupled......	450	60	24	300
Coupled......	450	60	20	360
Coupled......	450	60	36	200
Coupled......	450	60	16	450
Coupled......	450	60	12	600
Coupled......	500	25	24	125
Coupled......	500	30	20	180
Coupled......	500	60	32	225
Coupled......	500	60	26	257
Coupled......	500	60	20	360
Coupled......	500	60	16	450
Coupled......	500	60	12	600
Coupled......	500	60	6	1200
Coupled......	540	60	36	200
Coupled......	540	25	12	250
Coupled......	540	25	10	300
Coupled......	540	60	20	360
Coupled......	540	25	8	375
Coupled......	540	25	4	750
Coupled......	550	40	10	480
Coupled......	550	60	12	600
Coupled......	600	60	36	200
Coupled......	600	60	32	225
Coupled......	600	60	20	360
Coupled......	600	25	8	375
Coupled......	600	50	16	375
Coupled......	600	33	10	400
Coupled......	600	60	18	400
Coupled......	600	60	16	450
Coupled......	600	60	12	600
Coupled......	650	60	20	360
Coupled......	700	60	20	360
Coupled......	700	60	18	400
Coupled......	720	25	8	375
Coupled......	750	60	60	120
Coupled......	750	60	48	150
Coupled......	750	60	40	180
Coupled......	750	60	36	200
Coupled......	750	60	24	300
Coupled......	750	60	20	360
Coupled......	750	60	18	400
Coupled......	800	60	36	200
Coupled......	800	60	24	300
Coupled......	900	60	56	150
Coupled......	900	60	48	128
Coupled......	900	60	40	180
Coupled......	900	60	20	300
Coupled......	900	60	16	450
Coupled......	1000	60	44	163
Coupled......	1000	60	36	200
Coupled......	1000	60	28	257
Coupled......	1000	60	24	300
Coupled......	1000	60	20	360
Coupled......	1000	60	18	400
Coupled......	1000	60	16	450
Belted........	100	60	8	900
Belted........	110	30	6	600
Belted........	125	25	6	500
Belted........	150	60	12	600
Belted........	200	25	6	500
Belted........	150	60	12	600
Belted........	200	25	6	500
Belted........	200	50	12	500
Belted........	200	60	12	600
Belted........	250	60	20	300
Belted........	300	60	16	450
Belted........	300	25	6	500
Belted........	330	50	16	375
Belted........	425	50	16	375
Belted........	450	60	20	360
Belted........	540	25	8	375
Belted........	540	60	20	360
Belted........	550	40	10	480
Belted........	600	33	10	400
Belted........	600	25	8	375
Belted........	600	50	16	375
Belted........	690	60	20	375
Belted........	650	60	20	360
Belted........	750	60	24	300

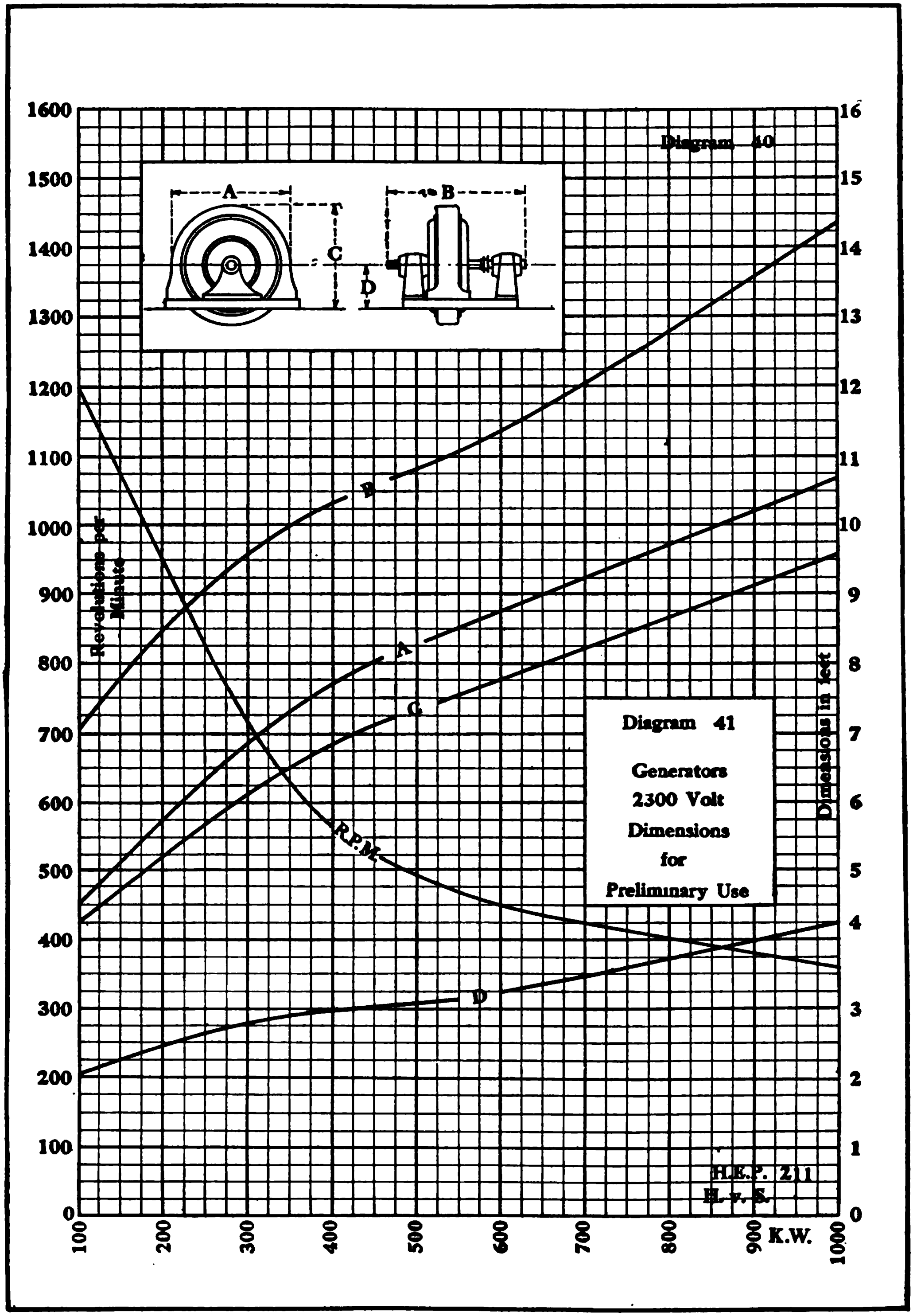


Diagram 40
A
B
C
D
Diagram 41
Generators
2300 Volt
Dimensions
for
Preliminary Use
Revolutions per Minute
Dimensions in feet
R.P.M.
K.W.
H.E.P. 211
H. v. S.
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1500
1600
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16

ARTICLE 101.—*The transmission plant equipment* consists of transformers, the line, and its terminal, the substation.

When the transmission voltage is to exceed that of the generator, which is generally the case with transmission distances exceeding fifteen miles, step-up transformers are required at or near the generating plant, since it is preferable to locate them in a separate room or even building from that where the generators are operating; and if the line voltage exceeds that at which the current is to be served, which will almost always by the case, step-down transformers become necessary at the terminal of the line, and the place where they are located is called the substation. It may be noted here that current service contracts are frequently made for the delivery of the current at line voltage to the customer's step-down transformer; this is especially probable when such delivery is of a large or major portion of all of the transmitted current, and if all of it is disposed of in this manner there is no need of a substation.

It is understood therefore that the transformer equipment may consist of step-up and step-down installations, or of the latter only, or that none may be required as a part of the power-plant equipment.

Transformers are of the same design and construction for either service, differing only as to the winding of the primaries and secondaries, as has been explained in Article 97.

The line consists of the supports, conductors, and fastenings.

The supports may be timber, concrete, or iron poles or posts, or steel-framed towers, their choice depending upon (a) the height of the conductors above the surface, (b) the length of the spans between supports, and (c) the cost of available material.

The line *conductor* may be of copper or aluminum. The weight of aluminum wire is about 0.47 that of copper wire of the same length and resistance, and when the cost of aluminum is therefore $1 \div 0.47 = 2.13$ that of copper wire or less the aluminum conductor will cost no more than copper conductor. The resistance of one mil foot aluminum wire is 17.0 ohms.

Some of the advantages of aluminum wire for the use of transmission line conductors are that sleet will not readily adhere to it, on account of its greasy surface; it is more economically transported and handled, on account of its lesser unit weight. Some of the disadvantages are that it cannot be readily soldered, on account of the greasiness of the surface,

and therefore joints are less conveniently made than with copper wire; also its surface is more easily injured in handling, dragging over rough ground or stones, because of its greater softness than that of hard drawn copper wire; and, as the melting point of aluminum is much lower than that of copper, there is more danger of its being fused by arcing across conductors.

The following table gives some of the characteristics of aluminum wire which are to be considered in connection with its use for electric transmission line conductors.

TABLE 33.—ALUMINUM WIRE.

Elastic limit 14,000 lbs. per square inch. Ultimate strength 26,000 lbs. per square inch. Resistance quoted is at 75° F. Resistance per mil foot = 16.949 ohms.

Size.	Area in sq. inch.	Lbs. per 1000 ft.	Feet per pound.	Ohms per 1000 feet.	Ultimate strength in pounds.
500,000 cm.	0.3930	460	2.041	0.03082	10,210
450,000 cm.	0.3540	414	2.415	0.03766	9,190
400,000 cm.	0.3141	368	2.718	0.04237	8,170
350,000 cm.	0.2750	322	3.106	0.04843	7,150
300,000 cm.	0.2360	276	3.623	0.05652	6,130
250,000 cm.	0.1965	230	4.348	0.06780	5,110
0000 B. & S.	0.1661	194.7	5.733	0.08010	4,320
000 B. & S.	0.1317	154.4	6.477	0.10100	3,430
00 B. & S.	0.1045	122.4	8.165	0.12740	2,720
0 B. & S.	0.0829	97.1	10.300	0.16050	2,150
1 B. & S.	0.0657	77.0	12.990	0.20250	1,710
2 B. & S.	0.0521	61.0	16.400	0.25540	1,355
3 B. & S.	0.0413	48.5	20.620	0.32200	1,075
4 B. & S.	0.0327	38.5	25.970	0.40600	852

The height at which line conductors should be strung will principally be determined by legal requirements, the general provisions being from 30 to 35 feet high when paralleling highways or crossing other aerial wire lines, and 20 to 25 feet when strung across country. It is therefore not always an economical programme to secure transmission line locations along public highways, as the cost of the higher supports may be much greater than the cost of private right of way across country, the latter affording the additional important advantage of guaranteeing control over the line and therefore making it practicable to protect it against interference from the public.

The length of the span is determined from the weight of the conductor and the consequential tension which is developed in its section, and the effects due to the temperature, wind, and sleet. Theoretically the sus-

pended wire takes the form of a catenary between supports, but it is altogether permissible to discuss the subject by considering it a parabola, whereby it is much simplified without the introduction of any considerable error.

If L is the length of the span, s the sag of the wire, its deflection from the horizontal, w the weight of the wire per foot length, then T the tension $= L^2 \times w \div 8s$ or $s = L^2 \times w \div 8T$; therefore the tension of a given wire varies inversely as the sag for the fixed span length, and for given tension and wire the sag increases as the square of the span length.

To prevent collision of the wires of a multiconductor line the sag should not be greater than twice the lateral distance between conductors.

The tensile strength of hard drawn copper is 17 tons.

The tensile strength of aluminum is 16 tons.

The expansion coefficient for copper wire is 0.0000096 per degree F.

The expansion coefficient for aluminum wire is 0.0000128 per degree F.

The sag of the span may be found from these values.

Example.—0000 copper wire is to be strung in spans of 120 feet length; then from above formula

$s = L^2 \times w \div 8T$, where $L = 120$, and w from copper wire table $= 0.64$ lb.

$s = 120^2 \times 0.64 \div (8 \times 5460) \times 4$; this latter factor represents the safety factor for normal conditions.

$s = 0.81$ foot or 9.75 inches.

This represents the minimum admissible sag in order to avoid raising the tensile stress above the basic safety factor of four, and from this the sag which is necessary to compensate for contraction due to low temperatures must be found.

If this line is to be constructed in the Northern latitudes, in Michigan, Wisconsin, or Canada, a low temperature—minus 20° F. or lower—must be provided for, and for this condition the proper sag is to be found from the actual length of the conductor in the span which is based upon the minimum sag as shown above. Thus,

$$Lw \text{ (the length of the conductor in the span)} = L + 8s^2 \div 3L,$$
$$\text{or } Lw \text{ for above example} = 120 + 8(0.81^2) \div 360$$
$$= 120.0146 \text{ feet.}$$

If we assume the normal summer temperature at 65° F., then the difference in temperature to be compensated for by increased sag will be 85°, and the contraction of the copper wire will be

$= 85 \times 0.0000096 \times 120.0146 = 0.098$ foot, which must be added to the length $Lw = 120.0146 + 0.098 = 120.1026$ feet.

The sag will be, from $Lw = L \times 8\,s^2 \div 3\,L$ or $s = \sqrt{3\,L\,(Lw - L) \div 8}$, for this case $s = \sqrt{360\,(120.1026 - 120) \div 8} = 30$ inches.

The safety factor in transmission line wire calculations should be adjusted to the climatic condition of the locality; if the prevalent wind movements are ordinary and no sleet storms to be expected, the factor of four above used is sufficient for practical purposes; where high winds prevail, such as should be credited with pressures of 40 and 50 lbs. per square foot, or if sleet storms are of yearly occurrence, the factor must be raised to six, and, in specially exposed locations, as for instance along the shores of the ocean or the Great Lakes, the factor should be taken at eight.

For aluminum wire the same method of calculating the span length and the sag applies, provided the proper values of weight, tensile strength, and expansion coefficient are substituted.

The ordinary practice is to make spans of a timber-pole line from 90 to 150 feet long; 106 feet is taken very commonly, requiring fifty poles to the mile.

Steel-framed towers are chiefly used when the transmission line is of double conductor circuits, which is always recommendable with high line voltage and large output plants; such towers are fifty feet and higher and the spans correspondingly longer.

Watercourses or wide swamps may have to be crossed on trestles. Submarine cables are rarely applicable in high-voltage transmission; they require an extra set of step-up and step-down transformers, as they are not reliable for any higher pressure than about 3000 volts.

Supports for a height of line conductors up to 35 feet may be of timber poles; they should be set one-seventh of their length into the ground, the buried portion and one foot above the surface being well tarred, and the top wedge shaped and painted. *Concrete poles* are now being constructed economically and make excellent line supports.

Steel-framed towers may be of various designs; some are shown on Fig. 134.

Conductor fastenings consist of cross-arms, insulator pins, and insulators.

Cross-arms (Fig. 134) are rectangular pieces of timber $3\frac{1}{4} \times 4\frac{1}{4} \times$ 4 to 5 feet long, or $3\frac{3}{4} \times 4\frac{3}{4} \times$ 6 to 8 feet long. They are preferably of yellow pine which is kiln dried and well boiled in linseed oil, and they are slightly rounded on top the better to shed the rain. Cross-arms are secured to the poles by being gained $1\frac{1}{2}$ inches and fastened with a $\frac{3}{4}$-inch round wrought-iron screw bolt passing through the cross-arm and the pole. The longer cross-arms should be further secured to the pole by two diagonal braces, which, for high voltage lines, should be of hard wood instead of iron straps. The upper cross-arm is placed about two feet below the pole top, and the others are spaced in accordance with the distance required between conductors, being from two to three feet.

Insulator pins (Fig. 135) form a very important part of the line. They carry the insulators, to which the conductors are secured, and are fastened to the cross-arms; they have to take up and resist all the lateral strain to which the conductors are exposed, and they also form the only available path for current leakage. Therefore they must be of suitable material, sufficient section, securely connected to their supports, the cross-arms, and should be well insulated. Insulator pins are preferably of oak or locust; they should not be of iron where high voltages are to be transmitted. Small section pins are to be avoided; they should be not less than $2\frac{1}{2}$ inches in diameter and 10 inches long, and as much heavier in section and longer as the weight of the conductor, which is to be secured to the pin, requires. Insulator pins must be thoroughly dried and well boiled in linseed oil. Pins are set into holes bored in the top faces of cross-arms and secured by a treenail driven through the shank of the pin and the cross-arm; spikes should not be used for this purpose. The top end of the insulator pin is threaded for about three inches to receive the insulator.

Insulators (Fig. 135) carry the conductor. They are made of glass, porcelain, or earthen-ware, consisting of one or more superposed bells (petticoats so called), their purpose being to shed the rain away from the insulator pin. Insulators should be tested for the voltage to be transmitted and for their breaking strength. The conductor passes over the

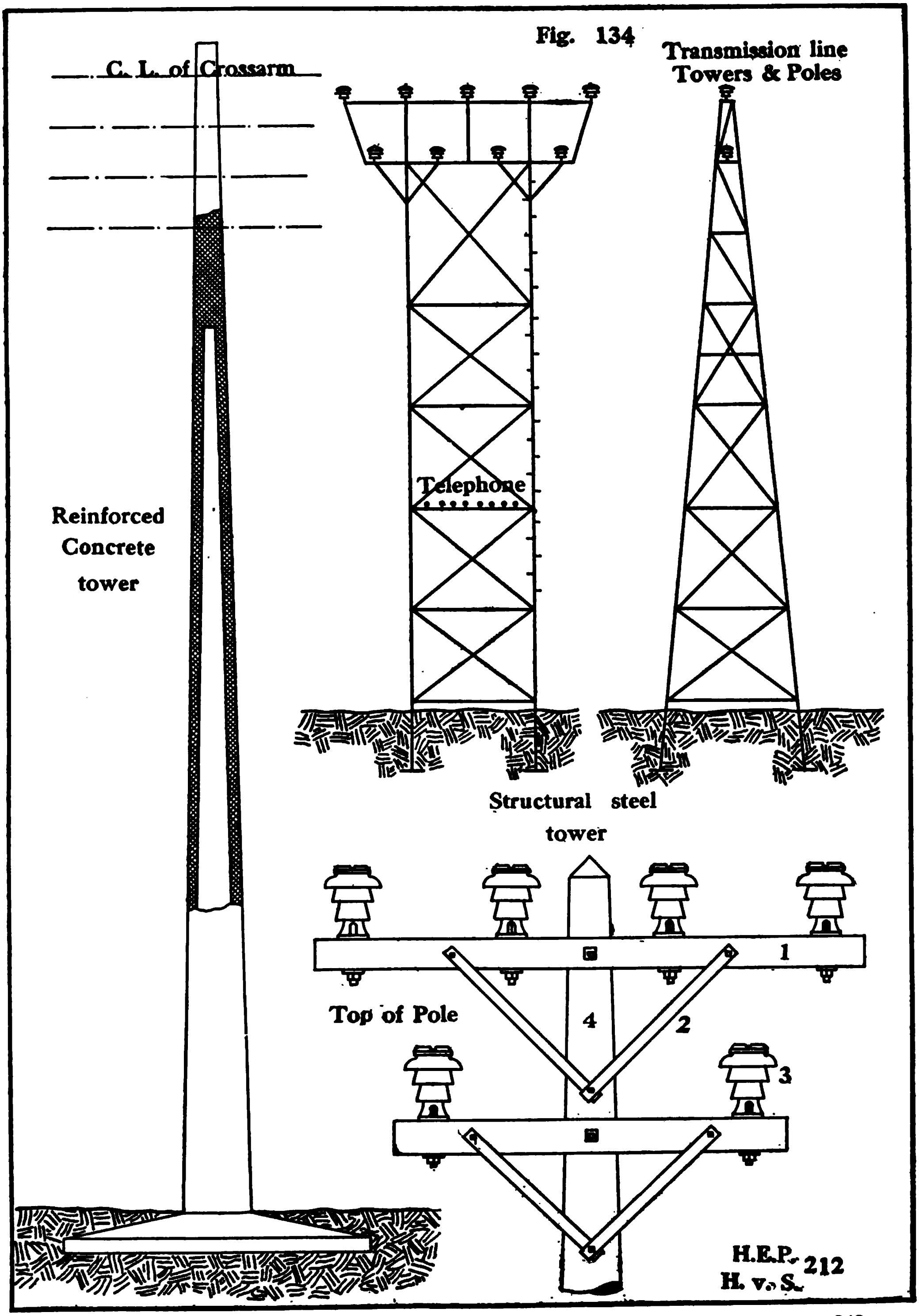


Fig. 134
Transmission line
Towers & Poles
C. L. of Crossarm
Reinforced
Concrete
tower
Telephone
Structural steel
tower
Top of Pole
1
2
3
4
H.E.P. 212
H. v. S.

top of the insulator, lying in a groove, and is secured to the insulator by a wire binding.

The substation is a suitable structure where the line terminates, and contains the required switchboard, the reorganizing and transforming equipment.

The entry of the line conductors into the substation must be carefully planned. It is generally effected by passing the conductors through insulator disks set into circular openings, of 12 inches or larger diameter, in the wall of the substation building, with some shelter over the point of entry to protect the conductors at the entry from rain and snow.

This closes the treatment of the electrical equipment of a hydro-electric plant, which has been necessarily very brief, as its detail discussion would assume the proportions of a separate volume.

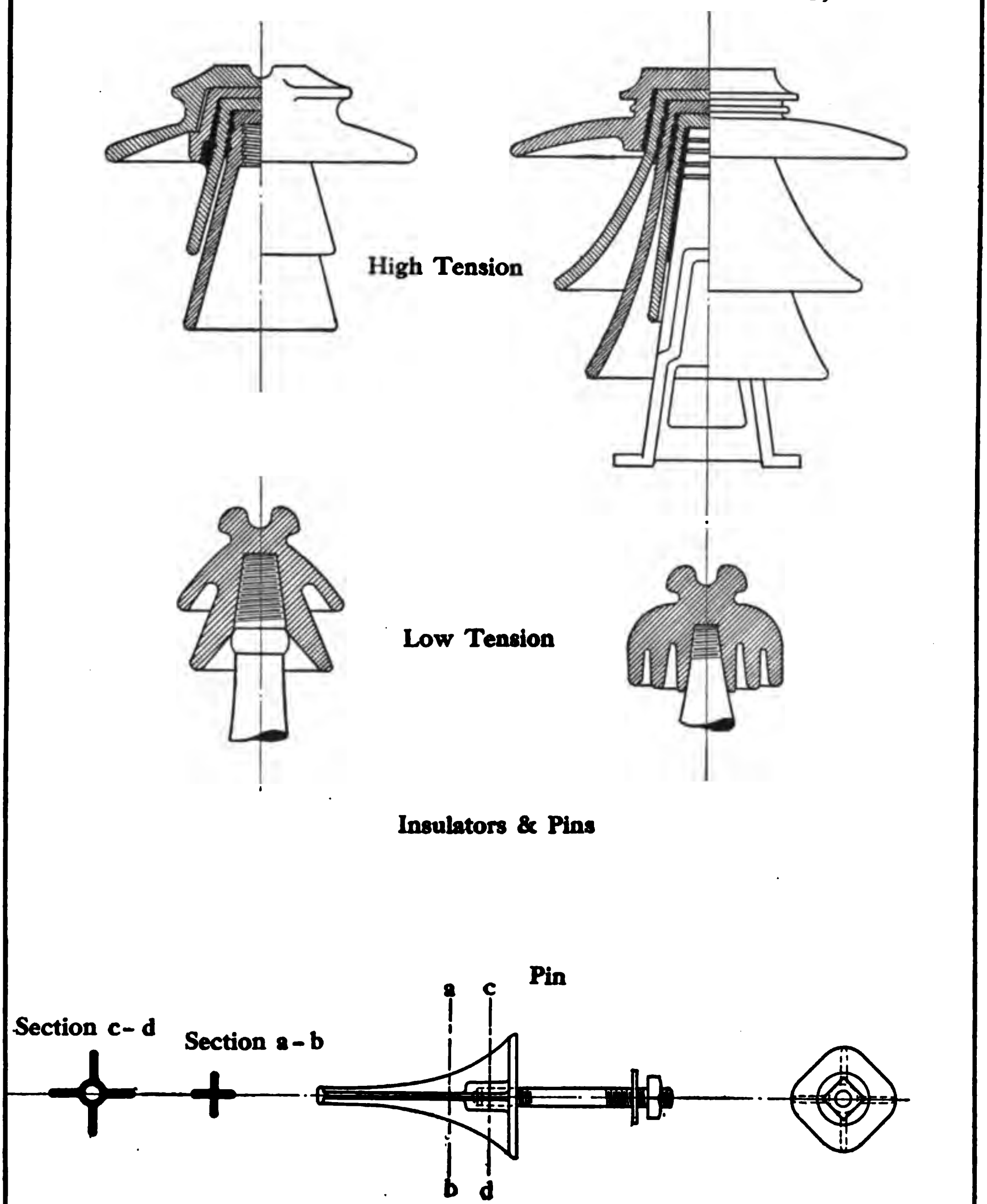
Figs. 136 to 140 give some general views of the partial installation of electrical equipment of the hydro-electric plant at Sault Ste. Marie, Mich.; the output capacity is about 32,000 kilowatts in 80 units of 400 kilowatts. Both continuous and alternating currents are being generated; the speed of all direct-connected generators is 180 revolutions per minute. This plant was designed by the author and constructed under his charge as chief engineer.

Fig. 136 shows the sectional armature of a 400-kilowatt 3-phase 30-cycle alternator, and Fig. 137 the revolving field of the same machine.

Fig. 138 presents a fine modern specimen of a continuous-current 400-kilowatt dynamo coupled to the turbine shaft, and Figs. 139 and 140 give a general view of the partial generator installation. In Fig. 139 the first three machines are D. C. dynamos, the others are alternators; the switchboard panels are on the left, which was a temporary arrangement, as they were finally placed along the wall on the right upon elevated platforms; in the foreground of Fig. 140 stands a rotary converter.

ARTICLE 102. *Auxiliary Power Plant.*—A very good and learned friend of the author remarked once that he could always find a hydro-electric plant by looking for a smoke-stack near a river, and this is as it generally should be if the development is a complete utilization of the opportunity. It is only exceptionally that the net earning capacity of a hydro-electric plant cannot be materially enhanced by the addition of a supplementary plant; this indeed is only the case when nature has provided a well-balanced ratio of power functions by furnishing a con-

Fig. 135



H.E.P. 213
H. v. S.

stant volume of flow and non-fluctuating head, or by supplying the means to maintain such an equilibrium in the shape of sufficient reservoir capacity. It is precisely the degree of deficiency of the natural supply, or of facilities to accomplish this, which represents the utility of the auxiliary plant, provided always that a demand for the current exists.

It is not the intention to go into any of the details of this topic here, as to the character and the make-up of the auxiliary power plant, whether it should be a plant of steam boilers and engine or steam turbine, or whether oil or gas engine, this forming a topic of operation rather than equipment. Here it must suffice to remind the investigator that the design and estimate of a hydro-electric plant is generally incomplete without taking into consideration this feature of an auxiliary power plant. Therefore the power station should be located and designed with this probable future requirement in view. The capacity of the auxiliary should be prima facie of at least one generator unit, and in this respect may influence the determination of the unit question. The auxiliary plant will be rarely called for until the hydro-electric plant has been in operation some years, but in estimating upon the whole project and deducing the net earning capacity of the proposed enterprise this factor must be included as an item of investment, of maintenance and depreciation charge, and of operating cost.

The storage battery or accumulator should also be considered as an auxiliary power plant factor. It consists of two inert metal plates, or of metallic oxides, which are placed in glass, earthen-ware, or wooden receptacles holding an electrolyte, which is a compound liquid separable into its constituent *ions* by the passage of electric currents through it. A storage battery is a collection of such *elements* chargeable by continuous current, which produces decomposition of the inert electrolyte between plates, whereby *cathions,* electro-positive radicals, are deposited on the plate which is connected with the negative pole of the charging generator, and *anions,* the electro-negative radicals, on the plate which is connected to the positive pole of the charging source. When the terminals of such a storage battery which is charged are connected outside of the electrolyte, an electric current is set up, flowing from the plate on which the positive radicals are deposited to that of the negative radicals, which is in direction opposite to that of the charging current. It may be noted here that it is erroneous to speak of the storing of the electric current, since the charging current is simply the means of and initiates

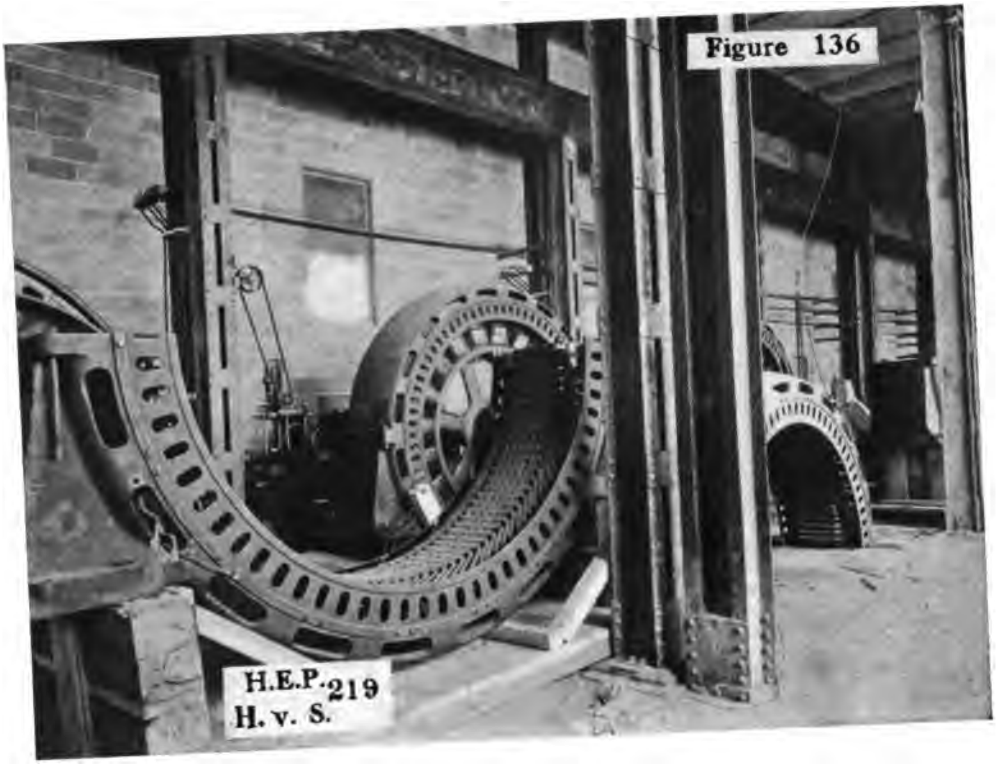


Figure 136
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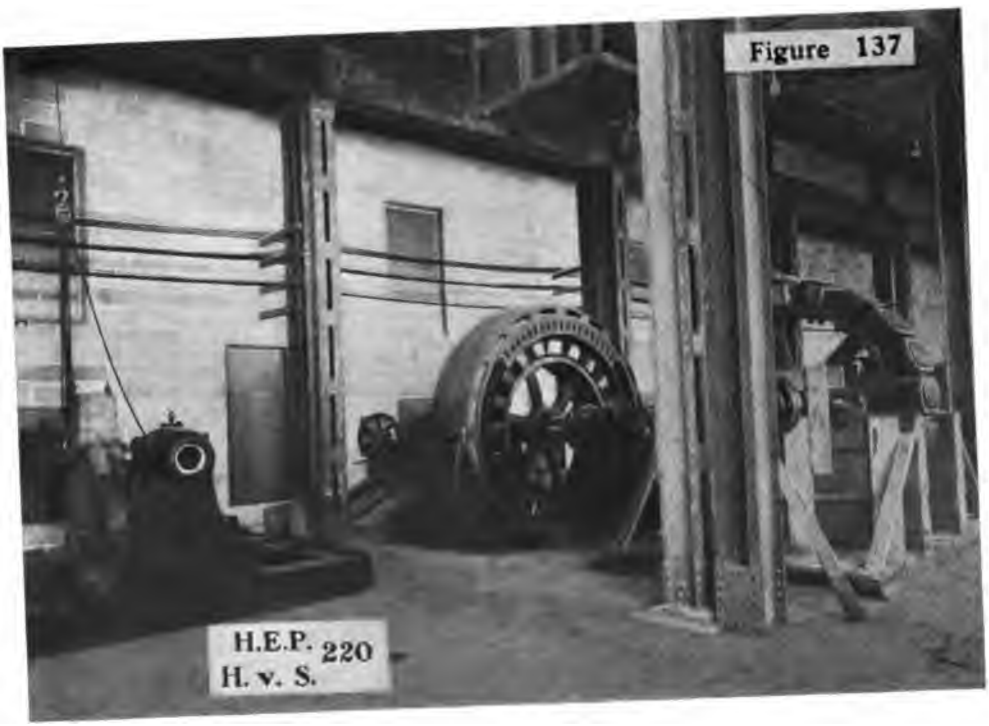
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Figure 138
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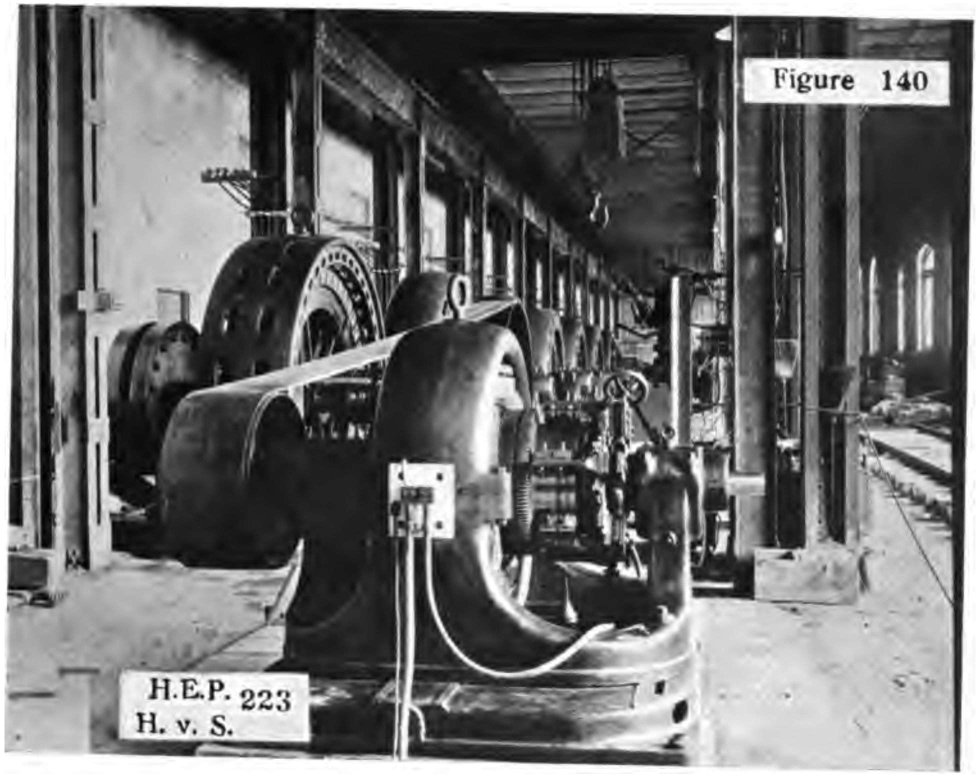

Figure 140


Figure 138
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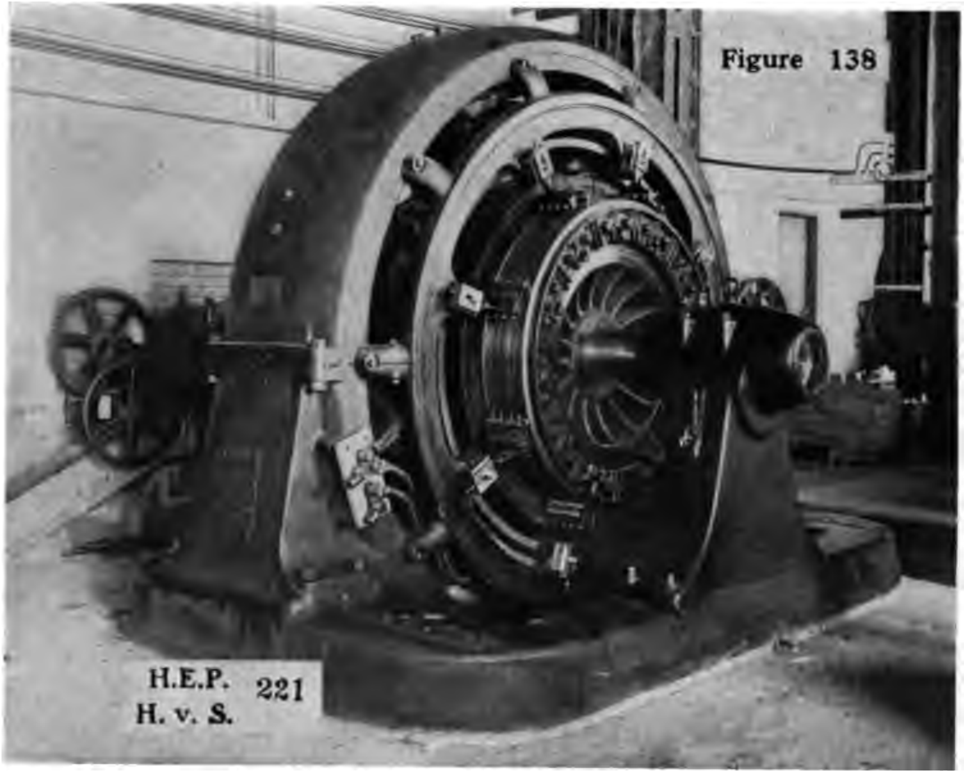
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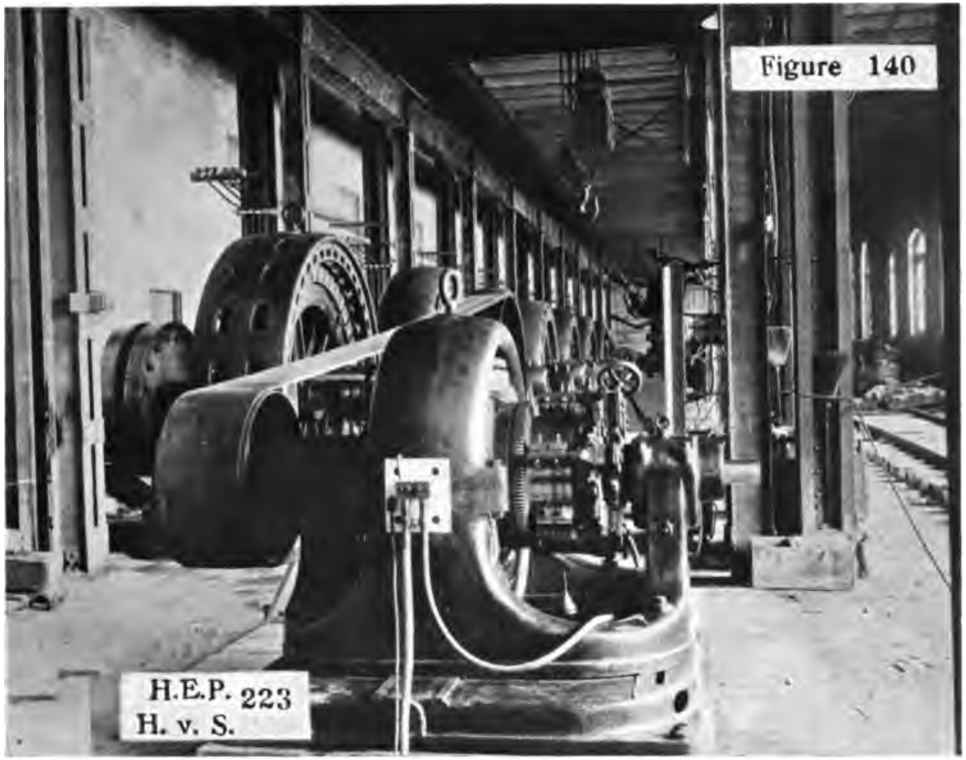
Figure 140
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the setting up of the battery current caused by the decomposition of the electrolyte, as above briefly outlined. Storage battery elements ordinarily represent electro-motive force of from 2½ volts up, but their range is very limited. They may be charged by the surplus output of the hydro-electric plant and discharged to add to the generating plant output in supplying additional current to carry the maximum, *the peak*, loads, and in small installations, when night water storage is necessary during low-flow seasons to accumulate the necessary day-load water supply, accumulators of this type can frequently be utilized to take care of the part night current loads made up of lighting business. Storage plants are also used for regulation purposes, and may then be located at the generating plant or along the transmission line; when they are to carry part loads they are preferably placed at the substation.

The subject of auxiliary power plants is treated in greater detail in Part III., "Operation and Maintenance of the Plant."

CHAPTER X

CONSTRUCTING THE PLANT

THE burden of all that has gone before is encompassed in the topic of this chapter, the realization of the development, and it does not seem essential to the treatment of the subject of hydro-electric practice to go very far either into the generalities or the details of this topic, which for hydro-electric plants does not necessarily present features which may not be found in the construction of other works. However, something, the author feels, should be said of the preparations for this final step, the construction, and of some features which are perhaps peculiar to this particular class of structures.

ARTICLE 103. *Plans.*—The proper plans for a hydro-electric plant are perhaps much more complex than those of any other single engineering undertaking, as they cover the wide range represented by hydraulics, hydrostatics, structures of timber, earth, rock, masonry, concrete, reinforced concrete, steel, hydrodynamics, mechanics, and electrical theories and practice; and they must necessarily be elaborate in details based upon calculations of functions, factors, dimensions, sections, etc., pertaining to all these various branches. Every pertinent detail should be designed to a sufficiently large scale readily to detect errors and, wherever practicable, algebraic calculations should be checked diagrammatically.

All calculations should be made in a computation book devoted to that particular project, and in a clear and precise manner, preferably in ink, each separate calculation of importance should be given a separate page, and when this book is completed with a comprehensive index it will prove a great labor and trouble saver in the future checking and even during the construction progress. All sheets containing plans should be of uniform size; dimensions should be given in figures and identified by dimension lines; the lettering should be plain; the author uses stamping machines for this purpose wherever practicable. Every structural feature should be shown in location, plan, longitudinal and transverse sections, followed by each important detail, and these latter should be numbered consecutively throughout the entire set of plans.

Originals should be inked before any of them are traced, which will obviate errors caused by the misinterpretation of pencil designs on the part of the tracer.

ARTICLE 104.—*Estimates* should never be made until the designs are completed and checked, and their original form should take the shape of a comprehensive detail tabulation, giving title of structure, number of detail and sheet, dimensions, quantities, and unit price of cost of material and of operation. The delivery cost of the material must be fully covered, as well as the insurance of construction plant, material, and of the personnel employed. A net profit of at least 15 per cent. should be added, and finally ten per cent. must be allowed for engineering and control.

Estimates should be analytical, as, for instance,

Concrete of 1 : 2 : 4 mixture:	
1½ bbl. Portland cement, deld., @ 2.25 per bbl.	$3.38
½ cub. yd. sand, deld., @ 1.00 per cub. yd.	0.50
1 cub. yd. broken stone, deld., @ 0.75	0.75
Mixing by machine	0.30
Conveying to site by barrows	
Placing concrete, monolithic, or	
Placing concrete in forms, at respective price	
Timber forms, used three times, cost of lumber deld., $30.00 per 1000 ft. b. m. per cub. yd.	
Removing forms per cub. yd., according to dimensions of structure	
Finishing, if outside wall floor or facing.	

Every item should be treated this way in estimating its cost. And finally the estimate should be concluded by a summing up of all the different material required and grouped in accordance with proper classifications as to its character and cost.

ARTICLE 105. *Specifications*.—The practice in this respect is so chaotically diversified that it may almost be called an individual business. The author believes this subject should be approached as far as practicable from the view-point of the constructor, with a full realization of his position, if he is seriously inclined to make a proper tender. The principal purpose, it seems, therefore, is to convey clearly the ideas and intentions of the designer of the plant. The structures of a hydro-electric plant differ considerably from those of like general character for other purposes. For instance, a diversion canal appears to many constructors not at all different from any other kind of water way, merely to pass the water, while it is, or should be, designed in accordance with definite scientific principles, and must be constructed in strict conformity to these in order

that the desired results be fully realized. So it is with the power station, which appears to be simply an ordinary kind of building, while in fact it is one of very peculiar and important details not met with in other building structures of even much heavier masonry sections. Earth embankments seem so much like those so common in railroad construction that it is an exceedingly difficult task to create the proper impression of their absolute specific purpose and therefore their entirely different construction. And so on along the line from the first to the last, and for these reasons the specifications for a hydro-electric plant cannot be too specific in clearly conveying their purpose, followed by the explicitly definite detailing of the structural methods by which these results are to be secured.

It is also well to bear in mind, when one prepares specifications, that the main purpose is to get this plant constructed, and to have this done as expeditiously and economically as can be, and have it done well, and that such results can only be hoped to be secured by a complete co-operation between the constructor and the engineer looking after the interests of the owners. It is absolutely useless in this respect, in the author's judgment, to burden the specifications with restrictions of the constructor's freest latitude of utilizing his own experience and ingenuity for the purpose of securing the best, speediest, and most economical realization wherever practicable. Methods so specified, but made applicable or not solely upon the dictum of the engineer, or elastic conditions depending upon the future interpretation or decision of the engineer, are calculated only to inject costly uncertainties into the undertaking,—that is, costly to the owners of the plant, not to the contractor, who is forced by all considerations of self-protection to discount them heavily, as it is not his province to take chances.

Quantities should always be quoted in positive figures; if they are uncertain the specifications should so state, and the probable fluctuations should be provided for upon a fixed and equitable basis of values.

The majority of specifications contain severe penalties for default in completing the works within a specified or agreed-upon time limit; few, however, provide any reward for anticipation of such a limit. Again, it is not to be expected that the enterprise is to be financed by the contractor, who should receive such a percentage of his earnings that they will meet his actual outlay.

These are generalities, nor can the subject be treated in any other manner,—that is, no hard-and-fast form can be defined as adapted to any certain range of conditions.

As to details, the author's meaning is best illustrated by an example.

Specifications of the Anchoring of a Spillway in Rock Location.

1. The site of the spillway is shown on Plan 3 and the dimensions are as thereon given.
2. All elevations are referred to Bench mark "D," being the top of an iron bolt 2 inches in diameter, which is set and leaded in the top of a granite boulder on the west side of the river on the north line of the spillway location, as shown on Plan 3, and about 15 feet from the crest of the natural bank; the elevation of this bench is 654.76 ft. above mean tide, New York.
3. So much of this area as can be conveniently coffered against the water at one time is to be entirely freed from all water, and is to be maintained in this condition at all times until the constructions described in this article, "Anchoring the Spillway," are fully completed, inspected, and accepted by the engineer.
4. The methods of coffering and of maintaining the dry condition may be as elected by the contractor.
5. The area is to be cleaned of all vegetable and earthy substances, and all loose rock or such as can be dislodged from the ledge by the ordinary use of a 12-lb. miner's pick, which is to be removed, and the material thus taken up may be disposed of as the contractor elects.
6. Anchor holes of the size and to the depth shown in Detail 22 on Sheet 11 are to be drilled into the rock bed and freed from all loose stone: they are to be spaced as shown in location Plan 3.
7. Anchor bolts of 3-inch round wrought iron 4 feet long are to be set in the anchor holes, the bottom six inches of the bolts being split open by one cut and spread to a diameter of $3\frac{1}{2}$ inches. Grouting, consisting of one part Portland cement, two parts of sand, and three parts of fine gravel, is to be poured into the hole while the anchor bolt is being held at the bottom and in the centre of it.
8. The anchor bolts, cement, sand, and gravel, and the mixing of the grouting are to conform to the specified material and method as given in the second article of these specifications.

In other words, there is no operation so unimportant but that it deserves to be clearly analyzed and so described in the specifications.

ARTICLE 106.—*Engineering control* of the construction of this kind of a plant cannot be any too thorough; short-sighted economy in this respect may be exceedingly expensive in the end.

All material should be inspected and tested; each operation should be carefully overseen; and, no matter how small the plant, the author has always found it justifiable to keep a daily progress record of each separate construction feature on specially prepared forms, of the time performance and therefore the cost; such a record is as valuable to the contractor as it is to the owner and certainly to the engineer. Nor should the camera be omitted in chronicling progress stages. Such a plant well constructed is a monument to all the parties concerned, and to the engineer who conceives and brings it into activity it should be a source of pride and satisfaction.

If the man who makes two blades of grass grow where formerly was only one is entitled to the plaudits of mankind, how much more is he who harnesses the now wasting energy of falling water!

PART III

OPERATING AND MAINTAINING THE PLANT

It was the original intention of the author to treat this topic in a separate volume, but this has been inadvertently delayed, and, as the rapid sale of the first edition now calls for some revision at a time before the first-planned program has been carried out, it is deemed advisable to abandon that and instead incorporate this subject in this treatise, and thus make it a complete presentation of all the topics pertaining to hydro-electric practice. In this way one volume covers all which is connected with the investigation of the project, the designing and construction of the development, and the final operating and maintaining of the plant.

The *operations* of a hydro-electric plant deal with control and utilization of hydraulic, mechanical, and electric forces and the part which each of these plays in realizing the ultimately desired results should be clearly and logically comprehended and appreciated by those to whom the operating supervision is entrusted. Every part of the hydraulic works and of the mechanical and electrical equipment must be put to its intended purpose for the highest obtainable efficiency, and maintained in that condition which will make this possible with the greatest economy of operating and maintenance costs. A thorough familiarity with all details of construction of works is as necessary as of the generating machinery; defects in dams, embankments, gates, conduits, penstocks, etc., can be appreciated properly only by the person who is intimate with their design and construction, and who may therefore understand the conditions of the invisible parts as well as of those in plain view. Without such knowledge maintenance could be applied only by conjecture, which cannot be expected to lead to economies but to the contrary. Operating and maintenance are practically, or should be, synonymous, at least when operation is qualified as of highest economical efficiency, and these two will herein be jointly treated first of the works and second of the equipment.

CHAPTER XI

The works comprise all the structures and devices which empound and control the water and conduct it to and from the turbines and those which house the equipment. They may be reservoirs, embankments, reservoir dams and outlets, power dam, pond, waste sluices, gates, fish-ladder, flashboards, intake, canal, flume, pipe line, forebay, tailrace, penstocks, power station. Each of these exists for a distinct purpose which must be fully realized in order that the best results are obtained from the plant; any serious defect in any of them will either endanger their safe stability or cause some loss of one or of both the power sources, flow and fall, and their prevention or correction is to be aimed at and successfully accomplished by operation and maintenance. The operating of these works embraces those measures by which the constancy of the available flow and fall is secured, the manipulation therefore of the provided flow and head control devices, while maintenance is concerned with their condition as to that state of perfection which is calculated to secure the best service, which means the preventing, or prompt repair, of any damage or defects.

Article 107.—*Reservoirs* may be of two distinct types, natural lakes or artificial ponds. The works of the first class may only consist of a dam and the outlet, unless the former level has been raised, when embankments or dikes may have been constructed across some low defiles; frequently such lakes have more than one natural outflow, which are closed when it is to serve as a reservoir. Artificial ponds are generally in the valley of some tributary or on the headwater streams, and are created by the closing of their natural outlet by means of a reservoir dam which is provided with a sluice-way. A telephone line connects the reservoir dam location with the power station.

Reservoir embankments are generally of the earth-fill type described in Part II, Article 73. The defects which are likely to develop are principally due to two causes, settlement and subsidences, and the first is followed by the second. Frequently it is considered unnecessary to place core walls in low embankments, and their omission may be expected to develop one or the other of these faults, which are permanently remedied only by the addition of the core wall or the substitution of an

impervious masonry wall on the water side of the embankment. Burrowing animals will do much harm to core-less embankments. When the embankment bed is not well prepared, not thoroughly cleared of stump and brush, settlement will take place and may cause slides. Tree planting offers the best security against subsidences which are likely to be caused by heavy rainfalls. The piling of sliding banks or balasting with rock are expedients of doubtful results, as the first disturbs the entire mass while the second may enlarge the sliding area because of the added weight. Slides are generally traceable to an excessive ratio of clay in the fill; when this is kept of the mixture specified in Article 73, they are not likely to be caused by rainfall. It is good practice to keep suitable filling material in conveniently placed supply banks at various points along the embankment for ready use in case of needed repairs. The embankment should be frequently inspected, at least monthly, especially following a heavy rainfall and in the spring after the ice breaks up; it is at this latter period that green embankments will most probably need some repairing, as the fluctuations in the reservoir level will pile ice-floes against the water slopes and cause more or less erosions. The feeder inlet to the reservoir must be kept free of sedimentation to the greatest obtainable depth; snags and log jams should be promptly cleared away. Vegetation should not be permitted to grow rank along the flowage level, this is best prevented by grubbing and cutting after the spring flood is down and again in the fall of the year.

The *reservoir dam* is generally an earth or rock-fill structure with contained masonry spillway and sluice. What has been said of the maintenance of the embankment applies equally to the dam; however, this is likely to be constructed with greater care and therefore will not so readily develop serious defects. The critical portion is the connection of masonry structure with and in the main dam where leakage may be looked for. Reservoirs are planned for the storing of water and undue leakage from them therefore defeats their purpose more or less; it represents just so much waste of the useful power source and may sooner or later grow to serious proportion unless it is promptly dealt with. In a properly constructed reservoir dam the embankment core wall is imperviously connected to the side walls of the sluice-way, and in that event leakage can pass only beneath the footings of these parts. When the embankment has no core and the sluice walls no wings passing into the embankment, nothing will prevent leakage to pass through and along

the sluice walls, and the only efficient remedy in such a case is to add a core or wing wall. When the leakage is below the footings, sheet piling along the upstream side may be resorted to with good effect.

The sluice-way should have a masonry or timber floor extending some distance up- and downstream of the gate openings, and when properly placed this will develop no defects.

The gates and operating devices may be of a great variety of types; they will require frequent inspection by actual operation; all metal should be kept from corrosion; gears, shafts, and racks should be coated with paint and tallow. Sediment should be cleared from the sluice entrance frequently. It is good practice to move a drag from a boat across the approach to the sluice while water is drawn, whereby the deposited silt is dislodged and carried through the sluice. A gauge reading to tenths of a foot should be placed near the upstream entrance to the sluice and its reading taken daily and entered in the Reservoir Log. The lowering effect upon the reservoir level caused by various gate openings during fixed periods of time should be observed on the gauge whenever opportunity offers and records made of it in the Log. Heavy ice should be prevented from forming in the sluice entrance by the cutting of channels along sluice or pier walls and upstream of gates. If gates move in channels ice should be prevented from clogging these by the liberal use of salt and all exposed operating gears should be housed during the winter season.

Article 108.—*Pondage* defines the accumulation of water upstream of the power dam, which is generally the deepened and widened river channel, the water spreading over the otherwise normally dry river bottoms, up into ravines, tributaries, and into low marshes and meadows. The pond is not confined by artificial embankments unless this is necessary at certain points to prevent its spreading upon highways or non-controlled lands. The defects which may diminish the intended utility of a pond are caused by sedimentation and ice. The remedy for both lies at the dam and whatever withdrawal provisions have there been made and what has been said of reservoir maintenance in this respect applies also in the case of the pond.

The function of the pond is not unfrequently erroneously accepted as being that of a reservoir. That a development possesses a valuable safeguard against a low water supply because there is a large pond above the dam is a very common but often ill-founded belief. Even large ponds,

as appearing from the great distance to which the water is backed, represent insignificant areas when analyzed for storage capacity. A pond ten miles long confined between the high-water river banks with an average width of 250 feet has an area of

$$\frac{5280\times10\times250}{43560}=305 \text{ acres}$$

and if the operating head is twenty feet it can be lowered only two feet without reducing the turbine and generator speed so much that the voltage drop and consequent current output loss will practically offset the power increase from pondage flow. Drawing this pond two feet aggregates a supply of 610 acre-feet or continuous flow

of 305 cubic-second-feet for 1 day
of 61 cubic-second-feet for 5 days
of 30 cubic-second-feet for 10 days.

In fact pondage supply is available only for a plant which does not operate continuously, such as the mill power so called. For a plant producing electric current to serve a mixed day and night power and lighting load, the pond does not represent a supply source of any stability, but rather of weakness, as in the event of its not refilling promptly the effective head remains reduced until the natural supply from the drainage area reinstates normal conditions. Only when the head is high and pondage area goes into thousands of acres does the pond partake of the utility of a supply reservoir.

ARTICLE 109.—*The dam* is the gateway and key to the power plant and frequently its most important and costly part, and for these reasons it is generally planned and constructed with that care which guarantees for it safety and permanency. The likely defects in a dam are leakage under its foundation or around the abutments, and their treatment is much like that described for the reservoir dam. When leakage develops under a dam which is founded on rock, a remedy is not so readily found. The locus of the leak must first be determined, and is to be looked for through a rock stratum with its heading in the river bed at possibly a considerable distance upstream. To locate the origin of the leak it will become necessary to lower the pond, when an examination of the shallow flowing water will reveal the point by the bubbling of the water into the

rock crevice which marks the upstream opening of the defective stratum. When found, one of several treatments may be employed to fill the fissure with some impervious matter, preferably cement grouting, which must be pumped into it with some pressure. Water flow must of course be excluded for a sufficient time to permit the filling to set. Small leakage under the dam which is founded on alluvial material may be arrested by depositing ashes along the upstream face, which will be washed into the openings and may close them. When this is not effective, or when it is a leak of some proportion, sheet piling may be resorted to, and if this fails to correct the defect the pond must be drawn down and the concrete cut-off wall lowered below the leaking point. The superstructure will develop leaks only when cracks open in it because of settlement or temperature influences. Leaks around abutments are probably more frequent than in other locations, because abutment and embankment cores are not carried to sufficient depths or are founded on porous material. The remedies are those above outlined. Thick ice must not be permitted to form against the dam where open channels should be maintained. A properly designed dam contains some submerged sluice capacity through which silt may be drawn from the upstream face and pond. Modern dams are generally equipped with devices to control head and flow, the former by the temporary raising of the normal pond level by means of *flashboards* of various types, and the latter by affording the flood waters the largest possible passage way through *sluices*. All of these operating devices must be kept in the best of order at all times and must be operated at frequent intervals to test their efficient condition. When there is much floatage, such as logs, brush, and ice, a *diversion boom* should be swung diagonally above the dam directing this floatage to the end opposite the diversion intake or power station, where a suitable sluice or spillway section should be arranged to pass it into the lower pool out of harm's way as far as other works of the plant are concerned. In conclusion of dam maintenance, especial attention is called to a feature which is almost universally overlooked,—namely, the condition of the *apron* downstream of the dam and of the river bed at the termination of the apron. This part receives the overfalling water and floatage and may be seriously damaged during the short period of one exceptionally high flood passage by being ruptured in places, which may soon cause its complete washing out. It should be examined after each such occasion, and the examination should be extended to the river bed at the lower end of the apron,

which is likely to be eroded, and if this is found to be the case the hole should be refilled with large pieces of stone.

ARTICLE 110.—*Diversion works* form a part of the plant only when the power station is located some distance downstream from the dam, and then consist of an intake, the conduit, which may be an open canal, flume, or pipe diverting the water from the upper pool, and the forebay.

The intake is a basin of larger flow section than that of the diverting conduit proper at which it terminates. Its office is to assemble the water, for the purpose of diversion, in a pool of considerable area, affording facilities to manipulate floatage which may enter in order to prevent its passage into the conduit and to reduce the flow velocity to a minimum. It may be confined between natural or artificial embankments or walls, and all that has been said as to maintenance of the reservoir applies to the intake. Unless large in area it is advisable to keep the intake free of ice, and for this purpose some spillway should be arranged over which it can be passed without its entering the conduit. A submerged sluice near its terminal serves to draw off any silt which may find lodgement on its bed.

The *open-canal* conduit may be between natural banks or walls, its bed and sides in soil, rock, or masonry or timber lining. Water-tightness is the principal virtue of the canal, and leaks must be promptly stopped; the remarks relating to reservoir banks apply here. It is good practice to arrange a spillway at the terminal of the conduit, where it merges into the forebay, affording a last opportunity to pass off floatage and ice. The entrance of the canal is formed by *headgates* controlling the inflow, and *trash-racks* may be placed upstream of these to intercept all small floatage. Head-gate maintenance involves all that has been mentioned for reservoir sluice-gates; they should be frequently tested, especially during the winter season. Modern practice generally provides electric gate-operating devices. The *forebay* is an enlargement from the conduit at its terminal for the purpose of lowering the flow velocity and distributing it to the turbine chambers of the power station. A trash-rack is sometimes arranged in the throat of this forebay, though one may be provided for at the head-gates.

The diversion conduit may be an open *flume* of timber, masonry, or steel construction. Its entrance is formed in a masonry or timber bulkhead where the head-gates are placed. A flume is generally wholly or partially supported on timber or steel trestles or masonry benches. The support foundations are responsible for the permanency of the shape

of the flume and therefore largely for defects causing leaks. Flumes must be kept free from ice to prevent rupture and should be provided with frequent spillways or sluices for ice passage. Before leaving the subject of open conduits the subject of ice, already so frequently mentioned, calls for a final word. So far only the surface or cake ice has been referred to, but, when the conduit entrance is in the vicinity of rapidly flowing water, *anchor* and *frizzle ice* may be formed. The origin of these is needles, and flakes which accumulate in large masses grow in volume like a snowball rolling down a snow-covered hill and when reaching the trash-racks will obstruct the passage of water. The most effective remedy is a low entrance velocity, while movable trash-rack sections make it possible to dislodge and break up the accumulations. If sunlight is kept from water surface, anchor ice cannot form, but this is not practical on large-area conduits unless they are permitted to freeze over.

Pipes are the last type of diversion conduits and are then known as pressure lines. These may be of wooden-stave, steel-plate, riveted or welded, and concrete-steel construction. The defects in pressure lines are chiefly leaks, sedimentation, and distortion of flow section. Leaks are due to the opening of joints, which may be caused by settlement or faulty riveting, and can only be effectively remedied by emptying the line and resetting of rivets. Sedimentation reduces the flow area, increases the velocity and thereby the friction head. Mud boxes should be arranged at intervals of 1000 feet to flush the silt out. Distortion of section is due to settlement of pipe and can only be remedied by withdrawing the water and reshaping, though this is not a serious defect. When pipe lines are laid with proper gradients, air cushions will not form; in the event that it becomes unavoidable to run lines with rises exceeding ten feet above the hydraulic gradient, air valves must be arranged at the peak of the rise and these should be frequently tested. Properly designed pressure lines are provided with stand-pipes of ample dimensions, and in that case the pipe will not be injured by water-hammer. All this applies to the different types of pipe lines, excepting that leakage from wooden-stave pipes is generally arrested by the tightening of the steel bands at the point of leak, while there is no remedy for leakage from concrete pipes. Metal pipe must be kept well coated, preferably with a good quality of graphite paint.

ARTICLE 111.—*The power station* is located at the dam or the terminal of the diversion conduit. In the first position it may receive the

water directly from the upper pool or from a forebay guarded by headgates. When taking water from the pond the power station stands on the dam alignment; when fed from a forebay it stands at some angle to the dam, the forebay paralleling it on the land side. In either position the power station is exposed to all the consequences of the fluctuating head and flow and, when on the dam alignment, to those growing out of floatage accumulations.

The power station rests upon a substructure whose upstream part, excepting when water enters in pipes, fills the office of a dam and is subject to the defects heretofore outlined for that structure. The superstructure houses the equipment and is not exposed to hydrostatic pressure. The power station proper is not likely to develop any serious defects unless there be leaks beneath or around it from the forebay, and their treatment must be by means of remedies already detailed. It is assumed that the walls and floors are properly designed not only to support safely the structure and equipment but also to be rigid against vibration.

The *pit* and *tail-race* are part of the power station, forming the exit passage of water discharged from turbines. The first is arranged in the substructure of the station, the second in the river channel below the power station. No serious defects are to be expected in either of these when, by proper design, they are of ample area and depth. When the equipment consists of several units, some of which are operated only for the generating of secondary power from higher than normal flow, it may be found that the tailrace under and from the idle units becomes silted, thereby reducing the depth under draft tubes to such an extent that the outflowing water backs up and reduces the effective head. The ready remedy is to operate each unit alternately to avoid long periods of idleness.

CHAPTER XII

The Equipment consists of the hydraulic, electric generating, auxiliary plant, and electric transmission. The first two are always housed in the power station, the third may or may not be, while a portion of the fourth is also there located. All equipment is generally subdivided into units, those of hydraulic and generating electric corresponding in capacity, while the auxiliary installation may conform to the hydro-electric unit characteristics or differ, which is also true of the transmission plant as far as it relates to the transformers. The maintenance and operation of the equipment is wholly of a mechanical character, and involves details which could be adequately treated only in a large volume, which will not herein be attempted; only those essentials which relate chiefly to the hydro-electric end of it will receive attention, with some general outline of the economical constitution of auxiliary power installation.

Article 112.—*The hydraulic equipment* consists of the turbines and governors. Reaction turbines are drowned, in open chambers or bays, or encased in draft chests, so called, into which the water is fed by pipes; shafts may be horizontal or vertical. In either event the most frequent troubles are caused by floatage, principally small blocks of wood which find their way into the turbine runner, blocking it or chipping its buckets. This is a common occurrence when lumber-mills are located on the stream above, and can be guarded against only by frequent raking of trash-racks, which should contain an especially close-spaced section from low to high flow surface. Reaction turbine gates are of cylinder, register, or wicket type, the last of which is more likely to develop leakages with time than the first two. Turbine gate shafts will develop defects before any other part by wear in the bearings, which have to be relined when this occurs. The causes of ice interference have already been mentioned; when anchor ice blocks the turbine gate opening, the safest method to secure relief is to close the penstock gate and disperse the ice accumulations by mechanical means or steam. If this is attempted by operating the turbine gates, it is quite likely that some of the gate riggings will be injured and the turbine put out of commission for repairs. On multiple

horizontal turbine lines the shafts may bind because of the uneven wear of some of the lignum-vitæ blocks; these are adjustable. If the stream carries much silt the runner blades will be ground off, thus enlarging the normal clearing between runner and case ring and increasing leakage; all means of intercepting sand before the water reaches the turbine should therefore be exhausted, as there is no remedy afterwards excepting runner renewal. Erosion and pitting of the runner blades is likely to develop when their design is faulty. When turbines are cased a pressure-gauge should be installed on the top of the draft-chest and its hourly reading recorded in the Log, so should the gate openings and speed be subjects of hourly measurements and entry and water-gauges should be maintained in open penstocks. Impulse wheels receive the water from nozzles and are free from injuries caused by floatage or from interruptions by ice. The buckets, or cups, have a short life compared with that of reaction runners, but are readily renewed. Turbines should be frequently examined, and, by keeping a careful record of gate openings, active head, and output, any considerable efficiency loss is readily detected, when the cause should be determined and corrected. The draft-chest, supply penstock, and draft-tubes should be kept well coated with a suitable paint.

The turbine speed is controlled by hydraulic or mechanical *governors,* which regulate the supply of water passing through the gates to the runner or, in some later types, the head by admitting air into the draft-tube. Governors should be adapted to the speed-regulation requirements of the plant, which are influenced by the character of the current service and load fluctuations. Governors require the same intelligent care as does any precisely operating machine.

ARTICLE 113.—*The electric-generating equipment* consists of exciters, generators, and the regulating devices. *Exciters* are direct-current motors required to excite or magnetize the field magnets. These may be driven by individual turbines or by belting them to the generator or turbine shaft; their speed is generally three or four times that of the generator. Exciters are readily maintained in efficient operating condition; their only parts requiring frequent attention are the commutator brushes which are adjustable.

The *generators,* which in hydro-electric plants are most generally of the alternator type, are not likely to develop defects, provided they are not overlooked beyond the safe heating limit. The armature is the

part which may become injured by overheating, and it is proper practice to keep in reserve an extra one at the station.

The regulating devices are assembled on what is called the *switchboard,* generally a stone panel, on which are mounted the instruments which measure and indicate the exciter and generator output characteristics; when properly installed these will require little maintenance attention.

The important feature in the operating of the electric-generating equipment, whence the current is transmitted to a distant point, is to secure the best constant harmony of output characteristics of the different units. This is accomplished at the starting of a unit by aid of the governor, whose speed-balls can be standarized for the desired regulation scope, but, as the load and the power-generating factors fluctuate, it requires more or less constant attention. All electrical equipments need "blowing out" at frequent intervals, for which purpose a suitable air-compressor should be installed, which is motor driven.

ARTICLE 114.—*Auxiliary power equipment* may or may not form a part of the hydro-electric station's outfit, but no station should be without some electric-storage capacity for station operating purposes, such as assisting in the regulation of the voltage on exciter outlets, operating of high-tension switches, and lighting the station. A storage battery will always prove a resourceful investment by taking care, in whole or part, of peak loads, and supplementing the exciter units in case of need. Storage batteries require constant operating care in charging and proper maintenance of their elements; they should be housed separately from the other equipment, where the temperature may be regulated by the aid of motor-driven fans. When neglected the depreciation of storage batteries may become high.

The location of a steam auxiliary should be decided largely by the conditions as to economical fuel supply, and in most cases these will point to the service rather than the hydro-generating end. The capacity of the auxiliary plant should be such as to guarantee the continuous power output which may be realized from the maximum hydro-motive equipment by supplementing deficiences caused by flow and fall reductions. The auxiliary plant units should always harmonize with those of the hydro-electric installation. The make-up of the steam-plant may differ greatly. The capacity being fixed upon, the steam consumption of the prime movers and auxiliaries must be determined in order to decide upon

boilers: 0.73 boiler horse-power are required per kilowatt output, and 10 square feet of water-tube boilers heating surface per boiler horse-power. Two boilers should be provided for each prime-mover and one extra prime-mover unit beyond the required capacity. The plant should be located convenient to ample water supply, and fuel tracks and fuel and ash handling should be planned for all obtainable economies. Steam turbines are the best prime-movers, those of horizontal type with coupled generator mounted on one bed-frame.

Cost of steam auxiliary plants and of their output is an important topic when the provision of an auxiliary is to be determined from such a searching analysis as has been outlined in Article 51, "Development Scope," and when efforts are made to convince steam-power users that it is much more costly than they believe and that the hydro-electric power current will bring a great saving to their business.

The steam-engine or water-turbine effective unit output is the mechanical horse-power which may be converted into electric horse-power with the loss of ten per cent. The commercial unit of electric energy is the kilowatt (1000 watts), and, as one horse-power equals 746 watts, a kilowatt represents one and one-third electric horse-power or about one and a half mechanical horse-power. The final commercial measurement of electric service is the kilowatt-hour, combining quantity and time of service, and this is therefore the proper comparative unit basis of power cost. The cost of power in manufacturing establishments is, as a rule, not definitely known to the operators; in most of them no doubt the total cost of the manufacturing product is conclusively established, but the segregating of the total into the component items, especially as relating to power, is only rarely sufficiently detailed. And it is therefore not to be wondered at that the actual cost, where generated by the plant, is greatly underestimated. The items of fuel, wages, oil, and waste are probably entered in the power account, but those of maintenance, repairs, depreciation, interest on plant's cost, taxes, insurance, etc., are rarely found properly apportioned. The following are the results of several years of investigations of first cost and fixed and operating charges of steam-plants of various mill capacities, which are believed to represent reliable cost estimates for present conditions and material and labor prices.

FIRST COST OF STEAM-POWER PLANTS.

(In dollars.)

	Boiler horse-power	50	100		300		500	
	Type of plant *	SNC	CNC	CC	CNC	CC	CNC	CC
1	Engines, boilers, and piping	3,000	5,500	6,200	12,700	14,500	20,000	22,500
2	Installation and accessories	500	800	1,000	1,500	1,600	2,000	2,500
1 and 2	Cost of equipment	3,500	6,300	7,200	14,200	16,300	22,000	25,000
3	Foundations, setting chimney	1,050	1,300	1,500	3,000	3,300	4,500	5,000
4	Boiler and engine house	600	800	800	2,000	2,000	3,000	3,000
3 and 4	Cost of building and placing	1,650	2,100	2,300	5,000	5,300	7,500	8,000
	Cost of plant	5,150	8,400	9,500	19,200	21,600	29,500	33,000
	Cost per boiler house-power	103	84	95	64	72	59	66

COST OF MAINTAINING STEAM-POWER PLANTS.

(Annual fixed charges in dollars.)

	Boiler horse-power	50	100		300		500	
	Type of plant	SNC	CNC	CC	CNC	CC	CNC	CC
5	Interest on first cost, 6 per cent.	309	504	570	1,152	1,296	1,770	1,980
6	Depreciation on equipment, 5 per cent	175	195	360	710	815	1,100	1,250
7	Depreciation on buildings, 2 per cent	33	42	46	100	106	150	160
8	Insurance, 1 per cent	51	84	95	192	216	295	330
9	Taxes, 2 per cent	102	168	190	384	432	590	660
10	Repair of buildings, 2 per cent	33	42	46	100	106	150	160
	Fixed charges, total	703	1,035	1,307	2,638	2,971	4,055	4,540
	Fixed charges, per horse-power	14.06	10.35	13.07	8.77	9.90	8.01	9.08

COST OF OPERATING STEAM-POWER PLANTS.

(Annual operating charges for 3300 hours.)

	Boiler horse-power	50	100		300		500	
	Type of plant	SNC	CNC	CC	CNC	CC	CNC	CC
11	Fuel in pounds †	8	5.75	5.25	4.5	3.75	4	3.5
	Fuel cost @ $2.00 per ton delv.	1,320	1,897	1,733	4,455	3,713	6,600	5,775
12	Oil and waste‡	135	200		475		675	
13	Personnel, one shift:							
	Engineer $3.00, fireman $2.00	1 E—0 F	1 E—1 F		1 E—2 F		1 E—3 F	
	Wages	900	1,500		2,100		2,700	
14	Repairs to equipment, 2 per cent of cost	70	126	144	284	326	440	500
	Operating charges, total	2,405	3,723	3,577	7,314	6,614	10,415	9,675
	Operating charges, per horse-power	48.10	37.23	35.77	24.38	22.05	20.83	19.35

* SNC = simple non-condensing; CNC = compound non-condensing; CC = compound condensing.

† Per boiler horse-power hour, with average load of 75 per cent. of capacity.

‡ No charge for water.

COST OF ONE STEAM HORSE-POWER PER YEAR.

Capacity in B.H.P.	50	100		300		500	
Type of plant	SNC	CNC	CC	CNC	CC	CNC	CC
Fixed charges	14.06	10.35	13.07	8.77	9.90	8.01	9.08
Operating charges	48.10	37.23	35.77	24.38	22.05	20.83	19.35
3300 hour horse-power	62.16	47.58	48.84	33.15	31.95	28.84	28.43
6600 hour horse-power	110.26	84.81	84.61	57.53	54.00	49.67	47.78

These represent the cost of steam horse-power when the plants are maintained in good order and run with normal efficiency loads, which presupposes that they are being operated with ordinary care and that the fuel is of good quality.

If the shop or mill is operated by mechanical drives, about 75 per cent. of the power plants output will remain available for actual machine work and the cost of power service would be 25 per cent. in excess of that of the generated power, or for compound condensing engines, in dollars, for

	100	300	500 horse-power
3300 hours	61.00	41.00	35.00
6600 hours	105.00	67.00	60.00

For larger plants the power cost will be reduced, being approximately for

	1000	1500	2000	2500 horse-power
3300 hours	30.00	26.00	23.00	20.00
6600 hours	52.00	45.00	39.00	34.00

This holds for $2.00 fuel and modern equipment.

ARTICLE 115.—*Transmission plant* consists of line and equipment. The maintenance of the line calls for its constant inspection. When the supports are steel towers, they need repainting, unless they are constructed of galvanized material. Cross-arms, pins, and insulators must be maintained at all times in best conditions, and damages from storms promptly repaired.

Transformers should be separately housed from the station equipment.

The substation forms the terminal of the transmission line, where the hydro-electric product is received, transformed, converted, and distributed for service. Its electrical equipment consists of step-down transformers, oil switches, converters, regulating devices arranged on switchboards, which call for the same intelligent maintenance as the generating equipment.

The operation of a hydro-electric plant should be carefully systematized in order to secure the best results with economy. It is logically divided between the three major parts of the plant, the hydraulic works, generating station, and transmission, with substation as its destination. A comprehensive system of records kept separately for each of these is indispensable to satisfactory and econominal results, and the old adage "a stitch in time saves nine" applies to no other industrial property more forcibly than to a hydro-electric plant.

It is poor economy to entrust a cheap man with the charge of so costly a property where a good salary may be saved for its owners by competency, system, and proper management.

Current service and rates are a fitting subject to close the treatment of the operating of a hydro-electric plant; and in this connection the reader will be best instructed as to the present-day practice of central power plants by citations from an official source. The following is quoted from a special report of the Department of Commerce and Labor, Bureau of the Census, 1910.

Regulation and Rates.—Various references have already been made in this chapter to the subjects of rates and regulation. It is well understood that, in their dealings with the communities served, central station companies have always been governed by the local-franchise ordinances under which they operated. But these franchises have dealt more with questions of public street-lighting than with such a feature as service to the private consumer; and it is in the latter respect that most change is noticeable of recent years. The change has been carried furthest in those States where public-service commissions exist, whose authority and control over public utility corporations have been generously amplified by the respective legislative bodies delegating such powers. These States are notably Massachusetts, Wisconsin, and New York, but it is significant that, as a matter of record, in almost every instance where the commissions have been appealed to, the actions or methods of the corporations have been sustained, or, if modified, the underlying principle has been adhered to as based on reason and equity. One of the most interesting recent cases is that in which the Wisconsin commission dealt with the application of the La Crosse Gas and Electric Company for the power to charge higher rates for electrical energy than had prevailed. The testimony and facts presented by the petitioner related mostly to the history of electric lighting in La Crosse, to the rates which the company was asking permission to establish, and to the various systems of fixed rates that were already in use. From the facts relating to the value of the plant and to its earnings and operating expenses, the commission said it was quite clear that the plant had not been a success as a producer of net earnings. This was especially true when some allowance was made for depreciation at 3 per cent. During the preceding two years the net earnings were not enough to pay any interest upon the investment nor even to meet ordinary depreciation charges, and so long as the rates charged for energy remained so low there was but little hope that the net earnings would increase. The decision included a discussion of one of the most important features of the problem,—the cost to the company of serving each class of customers. It is not necessary to cite here the rates fixed, but the language of the decision is as follows:

"It further appears that the proposed rates are somewhat lower than those charged in other cities, both inside and outside of this State. The comparisons we have made upon this point are quite extensive. They embrace at least 20 cities in Wisconsin and fully as many in other States. These facts are of considerable importance not only to the petitioner but the people who are served by this company.

The petitioner has duties as well as rights in this matter. While it is entitled to reasonable rates for service it renders, it has not the right to exact more than this. It must also see to it that the services it renders are adequate and that they meet all reasonable requirements in this respect. It is important that the interests of the public it serves should be as fully protected as those of its own. The best rates are those that are based upon the cost. Each customer should, under ordinary conditions, contribute his just proportion of all the expenses, as well as of the interest upon the investment. From the foregoing examination of the facts involved in this case it appears to us that the rates submitted by the petitioner fairly meet the situation, and that they are just and reasonable. It has been determined, therefore, that these rates shall be put into effect, subject, however, to such revision as may be found necessary when the plants in question have been appraised, or for other reasons."

At Minneapolis the city officials held that the rates of the Minneapolis General Electric Company were too high, and that the same rate per kilowatt-hour, except for quantity discounts, should be made for all customers without regard to conditions of load. The company had put in force a system of rates under which customers having the best load-factors—that is, those using current the largest number of hours per day—were given much the lowest rates. It appears from the reports of the early stages of the Minneapolis controversy that the city officials were chiefly concerned with lowering the maximum rates charged by the company for short-hour business. Several expert investigations were made into the company's affairs, with the result that the correctness of the company's theory of readiness-to-serve charges in connection with electric light and power business was upheld. The experts all agreed that the rates given to any individual customer should be dependent upon the fixed charges on the investment necessary to serve him, plus his share of the operating expenses necessary to serve customers in his class, rather than on the average expense of serving all classes of customers. However, as a concession to the smaller customers, it seems to have been generally agreed, both by the company and by the experts, that the maximum rates should be a little lower than those to which the smaller short-hour customers would be strictly and scientifically entitled. This reduction from the maximum rates to small short-hour customers was advocated only on the ground that the many small consumers, by the consent of whom the company had the use of the streets and public alleys for the distribution of its current, were entitled to receive compensation in this way for the franchise, and that larger consumers were not entitled to receive such compensation in the same proportion.

The Minneapolis General Electric Company and the committee of the city council came to an agreement on electric light and power rates as a groundwork for an ordinance giving the company a thirty-year franchise and fixing the rates for electric light and power for the first year of the franchise. The city council originally passed an ordinance requiring a uniform rate of 8 cents per kilowatt-hour, with discounts purely according to quantity. The company refused to recognize this ordinance, on the ground that it was unjust, inequitable, and confiscatory. The point of interest in the controversy is that a company was able to convince a council committee and citizens of the fairness of a rate based on load-factor, and of the unfairness of a uniform rate per kilowatt-hour for all classes of business.

The residence-lighting rate which was agreed upon is 9 cents per kilowatt-hour for the first fifty-two hours' use per month of 40 per cent of the connected load, and 6.66 cents for all over that. Commercial lighting is at the same rate, except that the maximum demand as measured by maximum-demand meters is substituted for 40 per cent of the connected load. Maximum bills are 100 per cent of the connected load. Minimum bills are $1 per month per lighting customer. Retail motor service pays 7.5 cents per kilowatt-hour for the first fifty-two hours per month of the customer's maximum demand, and 2.5 cents for all over that. The minimum bill is $1 per month per horse-power connected. The chief difference between these rates and the old rates of the company are that the maximum rate has been reduced on lighting from 12.6 cents for fifty-two hours' use of 60 per cent of the connected lamps to 9 cents for 40 per cent and the minimum bill on motors reduced from $2 to $1 per horse-power. Free incandescent-lamp renewals and free arc-lamp maintenance have been abolished under the new rates. Quantity discounts from 5 to 25 per cent are to be allowed on accounts of from $50 to $250 per month.

One of the experts employed in the investigations pointed out that light and power furnished under a limited-term franchise ought to cost the consumer more than that furnished under a perpetual

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franchise, because the company must figure upon paying off its bond holders and stockholders completely at the end of the limited-franchise period. A company could certainly float 4 per cent bonds on a perpetual franchise where with a limited franchise it would pay 5 per cent.

Rates were changed in one or two of the leading cities during 1907. The ordinance fixing the maximum rates to be charged by the Commonwealth Edison Company of Chicago until 1912 was passed by the Chicago city council on March 23 of the former year. This company pays 3 per cent of its gross receipts to the city, in accordance with the franchise previously owned by the Commonwealth Electric Company. The rates are as follows: Up to July 31, 1908, 15 cents per kilowatt-hour as a primary rate for energy used up to the equivalent of thirty hours' use of the consumers' maximum demand, and 9 cents per kilowatt-hour as a secondary rate for all energy in excess of the foregoing amount. From August 1, 1908, to July 31, 1909, the maximum rate is 13 cents and the secondary rate 7 cents. A discount of 1 cent per kilowatt-hour from the foregoing rates is to be allowed on all bills paid within ten days.

The Union Electric Light and Power Company, of St. Louis, has put in force a new system of rates, which differs considerably from the typical systems in use. It is founded on the belief that the value of the service rendered to any individual should, so far as practicable, be based on the cost of serving him, and not on the average cost of serving the entire body of consumers; and that, as the cost of supplying current per kilowatt-hour varies greatly with the different classes of service, so the price per kilowatt-hour, in justice to several users, should vary greatly to different customers. The company felt compelled to recognize the force of the argument of the customer who maintained that he was entitled to a lower average rate if he guaranteed $5 per horse-power per month than his neighbor who would guarantee only $1 per horse-power per month. At first a system of "special" contracts was adopted to meet this condition; but complaints of unequal discriminations led later to the substitution of a graduated schedule of rates. Under it the service is divided into a very much larger number of classes than was ever before attempted, and every consumer in the same class gets the same rate.

Each customer's rate is based on the minimum monthly guarantee he is willing to make per horse-power of per 50-watt lamp connected, and the rate is inversely proportional to the amount of the connected load. For example, the customer having fewer than 100 lamps pays 12 cents per kilowatt-hour if he guarantees only 10 cents per month per lamp. By guaranteeing 45 cents per month per lamp he gets a rate of 10 cents per kilowatt-hour, and by guaranteeing 65 cents per month per lamp, a rate of only 8 cents per kilowatt-hour. Of the customers furnishing the 10-cent guarantee there are 15 subclasses, each with its own modified rate. The rate also declines as the number of connected lamps increases. For example, a customer guaranteeing 10 cents per month per lamp and having less than 100 lamps pays 12 cents per kilowatt-hour. This rate is reduced by gradations until for 3000 lamps or over, with a 10-cent-per-lamp guarantee, the rate is 6 cents per kilowatt-hour. For the 45-cent-per-lamp guarantee, the customer with fewer than 100 lamps pays 10 cents per kilowatt-hour, while the customer with 3000 lamps pays 5.2 cents per kilowatt-hour.

All these rates are subject to discounts based on hours' use and quantity. The discount made according to the equivalent daily hours' use of the entire connected load starts with a 6 per cent discount for a kilowatt-hour consumption equivalent to one hour's use per day of the connected load, and rises by graduations to 25 per cent discount for a kilowatt-hour consumption equivalent to eighteen hours per day of the connected load. There is, also, in addition to this, a discount based on the amount of the bill, which is from 5 per cent. on bills of under $10 to 56 per cent. on bills of over $9,000 per month.

The motor rates are graded on the same plan. For a 1-horse-power motor customer they vary from 10 cents per kilowatt-hour on a guarantee of $1 per month per horse-power to 5 cents per kilowatt-hour on a guarantee of $7.50 per month per horse-power. The rate also depends on horse-power connected. Under the guarantee of $1 per month per horse-power the customer with over 500 horse-power gets a 5.5 cent rate. Under a guarantee of $2 per month per horse-power the rate is 4.5 cents. The rates for heating and cooking circuits in residences are 12 cents per kilowatt-hour on a minimum monthly guarantee of $2, 11 cents on a $3 guarantee, 10 cents on a $4 guarantee, 9 cents on a $5 guarantee, 8 cents on a $7.50 guarantee, 7.5 cents on a $10 guarantee, and 7 cents on a $15

guarantee. On these cooking rates a discount is given according to the quantity of current consumed; on bills of $5 or under 5 per cent is deducted, and this per cent increases by 1 for each $1 of increase in the bills up to $15, at which point the discount is 15 per cent. For bills of over $25 the discount is 20 per cent.

In its annual report for 1908 the Wisconsin commission said that it found the rates filed by the larger companies to be generally based on scientific considerations, but that those of the smaller companies partook of "every conceivable form and method of determination." Out of 119 companies reporting, 50 had no discriminatory rates, and 3 out of every 100 customers paid less than the schedule rates. The report went on to say: "Because a certain utility has more discriminations in effect than another does not mean in itself that it is following a vicious practice or is using unlawful methods. Most of the discriminations cited are remnants of a former period of unrestricted competition; others are the outgrowth of circumstances over which the utilities themselves have no control."

Both the Wisconsin and the New York commissions have sought to introduce a uniform classification of accounts for electric companies. Two sets of accounts are required in Wisconsin. In general, electric plants operating in cities of 10,000 inhabitants or over must keep at least the list of accounts prescribed in Class A, and all plants in cities of under 10,000 population must keep the accounts prescribed in Class B. Any changes or additions proposed by a company must be filed with the commission before the accounts in question are opened.

In New York City one of the features of the rate work of the public-service commission of the first district has been to make a more general provision for "break-down" service. It had placed the price of this service at $30 per kilowatt of maximum demand, against which the real consumption is an offset at regular rates. In other words, the commission has recognized the inherent propriety of a stand-by readiness-to-serve charge. The commission of the first district made an exhaustive investigation of the contracts made by the companies, revealing a negligible number of special contracts,—one or two hundred in scores of thousands,—and many of these, as in other businesses, left over from a former management or other control. One of the acts of the commission has been to prohibit specifically any "undue or unreasonable preference" or advantage "to anybody, while no charge shall be made that is not in a filed schedule, nor shall any electrical corporation refund or remit in any manner or by any device any portion of the rates or charges so specified." It is obvious that the immediate effect of such a general policy is to compel companies to classify their customers more closely, so that all in any given group shall be treated alike. The fundamental fact is that very few cases are alike in all particulars. Even where like conditions exist, sometimes the parties in question can not be persuaded of it, and the companies have insisted on the impossibility of meeting the rules of the commission either as to publishing every little concession to a customer's wishes or as to strict conformity with all the terms prescribed for contracts. A brief on this point filed with the commission by the New York Edison Company pointed out that one of its most important forms of contracts was for supplying energy to large buildings by wholesale or in bulk. These contracts were largely the result of personal canvass and individual negotiation, and it was claimed that if the company was not permitted to modify the phrasing or minor details of such contracts to suit peculiar conditions its business would be seriously interfered with. The company stated that it did not seek to make special terms or give unusual privileges to particular customers, but simply to be permitted to modify the contracts to suit different conditions. It desired only to extend to every customer any convenience or facility that the special conditions surrounding the service made practicable, provided that the peculiar features introduced into the contract did not modify the cost to the consumer, and provided that the company was prepared to extend the same privileges to all others who presented the same conditions. The company expressed itself as quite willing to accept and obey the order of the commission in so far as it prohibited any variation in charge, preference in rates, refunds, or special privileges; but it believed that special riders to the contracts with customers should be permitted to meet special conditions that did not affect the actual cost of furnishing the current, and it did not mean to discriminate in any way in favor of one customer as against another.

A valuable study of the whole subject of rates for electric energy is found in the decision of the Board of Gas and Electric Light Commissioners of Massachusetts in the matter of the complaint of the Public Franchise League against the Edison Electric Illuminating Company of Boston, filed May 29,

1908. In the opinion many of the points already discussed in this report, and others raised on the controversy, are given careful consideration. The Edison Electric Illuminating Company of Boston, like many other companies, has had a system of rates based upon "fixed costs" and "running costs," so as to charge each customer substantially the cost to it of supplying him, inclusive of a reasonable return on the investment, the basic method being known as the "maximum-demand" system. One of the various modifications of this system in use in America is the Doherty system, in operation in Denver and other cities. It is based fundamentally on the readiness-to-serve principle, and aims at a more or less exact adjustment of the price to the customer to the cost of producing that for which he contracts, and diverges widely from the idea of a uniform rate for all customers.

GENERAL INDEX